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Indiana Harbor and Canal (IHC) Dredging and Disposal Alternatives Analysis

Evaluation of Relative Disposal Requirements, Emissions and Costs for Mechanical and Hydraulic Dredging Alternatives

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Chicago, IL

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Acronyms

CDF	- Confined Disposal Facility
CMP	- Comprehensive Management Plan
DDR	- Design Documentation Report
ECI	- Energy Cooperative Industries
HELP	- Hydrologic Evaluation of Landfill Performance
HRT	- Hydraulic Retention Time
IHC	- Indiana Harbor and Canal
PSDDF	- Primary Consolidation Secondary Compression and Desiccation of Dredged Fill
RCRA	- Resource Conservation and Recovery Act
TSCA	- Toxic Substances Control Act
TSS	- Total Suspended Solids
WWTP	- Wastewater Treatment Plant

Preface

This report describes a study to perform a planning-level evaluation of a limited number of dredging and placement alternatives for the operation of the Indiana Harbor and Canal Confined Disposal Facility (CDF). Issues to be addressed include: 1) compatibility of hydraulic dredging or placement with the existing proposed CDF design, developed for mechanical dredging and documented in the Design Documentation Report (DDR) (USACE, Chicago 2000), 2) feasibility of expediting backlog dredging using hydraulic dredging rather than mechanical dredging, 3) relative air emissions (volatile and particulate) from the CDF for hydraulic dredging versus mechanical dredging, and 4) overall life cycle cost of hydraulic dredging versus mechanical dredging. The Environmental Laboratory (EL) of the U.S. Army Engineer Research and Development Center (ERDC) at the Waterways Experiment Station (WES) conducted this work. The U.S. Army Corps of Engineers (USACE) Chicago District funded ERDC under Project Order W81G6623056283. The initial project manager was Mr. Ajit Vaidya and the current project manager is Ms. Shannon R. Rose of USACE Chicago District.

This report was written by Ms. Trudy J. Estes and Dr. Paul R. Schroeder, Environmental Engineering Branch (EEB), Environmental Processes and Engineering Division (EPED), EL, and Mr. William R. Loikets and Ms. Elaine R. Taylor, Cost Engineering and Specifications Section, Technical Services Division, Chicago District. The WWTP cost estimates were prepared by Mr. Vivek Agrawal, Mr. Chris Caine, and Mr. Rich Gallas of Montgomery Watson Harza. Technical editing was performed by Ms. Cheryl M. Lloyd (EEB). Internal reviewers for USACE, Chicago District were Mr. John Breslin, Mr. Jan Miller, Dr. Jennifer Miller, Mr. Jay Semmler, Ms. Le Thai, Mr. Ajit Vaidya, and Mr. Jeff Zuercher. Independent Technical Review (ITR) members were Mr. Thomas Kenna and Mr. Paul Polanski, USACE, Buffalo District and Mr. Stanley L. Hanson and Mr. S. Edward Mead, USACE, Omaha District.

This study was conducted under the direct supervision of Dr. Richard E. Price, Chief, EPED, and under the general supervision of Dr. Beth Fleming, Acting Director, EL. Dr. James R. Houston was Director, ERDC, and Col. John W. Morris, III, EN, was Commander.

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1 - Introduction

Project Background

Indiana Harbor and Canal (IHC) is an authorized Federal navigation project located in East Chicago, Indiana (Figure 1). Project features include breakwaters at the harbor entrance and a deep-draft navigation channel (USACE, Chicago 1999). Channel depth ranges from 22 to 29 feet. Sediments in the IHC are contaminated and have been determined to be unsuitable for open water disposal, unconfined upland disposal or beneficial use. Dredging of the IHC has been deferred since 1972 while a technically and economically feasible and environmentally acceptable management plan was developed. As a result of studies undertaken by the US Army Corps of Engineers Chicago District to address disposal issues, dredging is to be undertaken throughout the IHC Federal navigation project to authorized project depths and widths. Dredging will also be completed in the appropriate berthing areas outside of the authorized channel limits at non-Federal expense to provide depths commensurate with those in the Federal channel.

The results of environmental studies and technical evaluations conducted in the course of developing a management plan for Indiana Harbor sediments are summarized in the Comprehensive Management Plan (CMP) (USACE, Chicago 1999), the Design Documentation Report (DDR) (USACE, Chicago 2000), and the Disposal Alternatives for PCB-Contaminated Sediments from Indiana Harbor, Indiana (Environmental Laboratory 1987a and 1987b). Design parameters and assumptions used in the present study were largely obtained from these documents.

The CMP is a two-volume report: Volume 1 – Final Feasibility Report and Environmental Impact Statement and Volume 2 – Technical Appendices. The CMP provides general project background, a description of plan formulation over several phases, the selected plan, and aspects of plan implementation, including discussion of disposal sites that were considered for the project. Three dredging plans were evaluated in the CMP (USACE, Chicago 1999). The first plan consisted of dredging the harbor and canal to authorized depths from the entrance in Lake Michigan to the E.J.E. Railroad Bridge (Reaches 1 through 5), plus the PCB hotspot along the north bank of Reach 6. This plan was identified as Alternative 1 - Partial Federal Channel Dredging. The second plan consisted of dredging the entire Federal navigation project to authorized depths from the entrance to the upstream project limits on the Lake George and Calumet River Branches (Reaches 1 through 13). This plan was identified as Alternative 2 - Complete Federal Channel Dredging. The third plan included the complete Federal channel dredging of Alternative 2, plus additional dredging provided for in a 1993 Consent

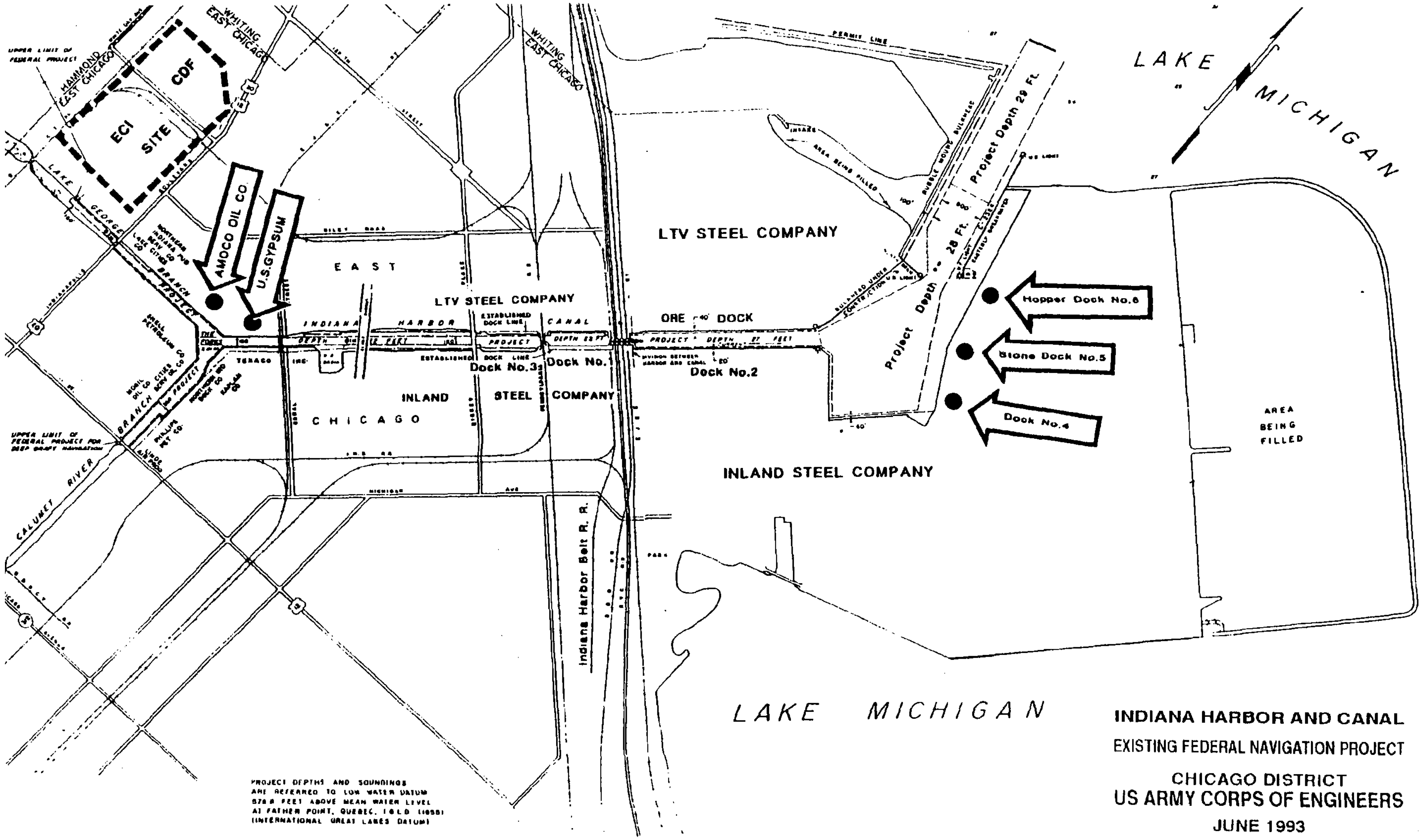

Figure 1. Indiana Harbor and Canal

Decree between the U.S. EPA and the Inland Steel Company. This plan was identified as Alternative 3 - Cooperative Dredging Program. All three plans include dredging in appropriate non-Federal dock/berthing areas to provide depths commensurate with the adjacent Federal channel depths. The selected plan in the CMP is the Cooperative Dredging Plan.

The DDR documents a design prepared for the selected plan from the CMP. The supporting technical analysis for hydrology and hydraulics, environmental engineering, geotechnical, structural, mechanical, and civil design along with a detailed cost estimate are presented in the ten appendices to the DDR. Based on previous examination of dredging technologies conducted during formulation of the CMP, it was determined that dredging would be conducted using a mechanical dredge, specifically a closed-bucket clamshell dredge. In the DDR, a projected dredging rate was established based on documented sediment depths and projected accumulation over a period of 30 years, and a design was developed for the selected disposal site.

Because it would be beneficial to expedite the backlog dredging, it was subsequently considered desirable to conduct a comparative evaluation of hydraulic versus mechanical dredging, prior to project implementation. This is the purpose of the present study; to conduct a planning-level evaluation and comparison to facilitate selection of the dredging method. Major project objectives include rapid dredging of the backlog sediments, while minimizing emissions and project costs. Some compromises may be required, however. For example, all water produced at the site as a result of dredging, consolidation of dredged material, precipitation, and groundwater management, must be treated before discharge. Minimization of water production would therefore be desirable from a cost perspective. However, backlog dredging may be best expedited using hydraulic dredging, which produces high volumes of water that must be treated. Minimization of volatile and particulate emissions is also considered desirable. Management of the facility to maximize dredged material drying and consolidation will facilitate the most rapid capping and closure of the site and will minimize the capacity of the storage facility needed. Particulate releases may be higher from dry surface materials however. These competing objectives must be weighed as a whole, and a determination must be made as to which alternatives and management procedures will best achieve project objectives at acceptable risk and cost.

The comparative analysis is based on the CDF design presented in the project DDR. Some aspects of the design may change during project development and feature design. The water concentrations and emissions calculations were based on a comprehensive analysis of the sediment conducted in 1986 (Environmental Laboratory 1987a and 1987b). The sediment quality in Indiana Harbor and Canal is highly variable. This data is not necessarily a complete characterization and may not be representative of all conditions. However, for the purpose of a comparative analysis using one comprehensive study provides a consistent framework for decision making. Several independent studies on various aspects of the CDF design and operation were underway at the time this report was being prepared. Information from these various studies was included only when specifically noted. This analysis is not intended to supercede or replace in-depth analyses on other project features. The calculations and design features presented in this analysis are intended for planning-level screening considerations and may not represent a final design or description of actual operating conditions. Dredging schedules, production rates, and other operational details are intended for planning-level screening purposes only. The actual dredging schedule may differ from that presented here. Specific project objectives and approach are more fully described in the following sections. The overall design process encompassed in previous and present efforts is illustrated in Figure 2.

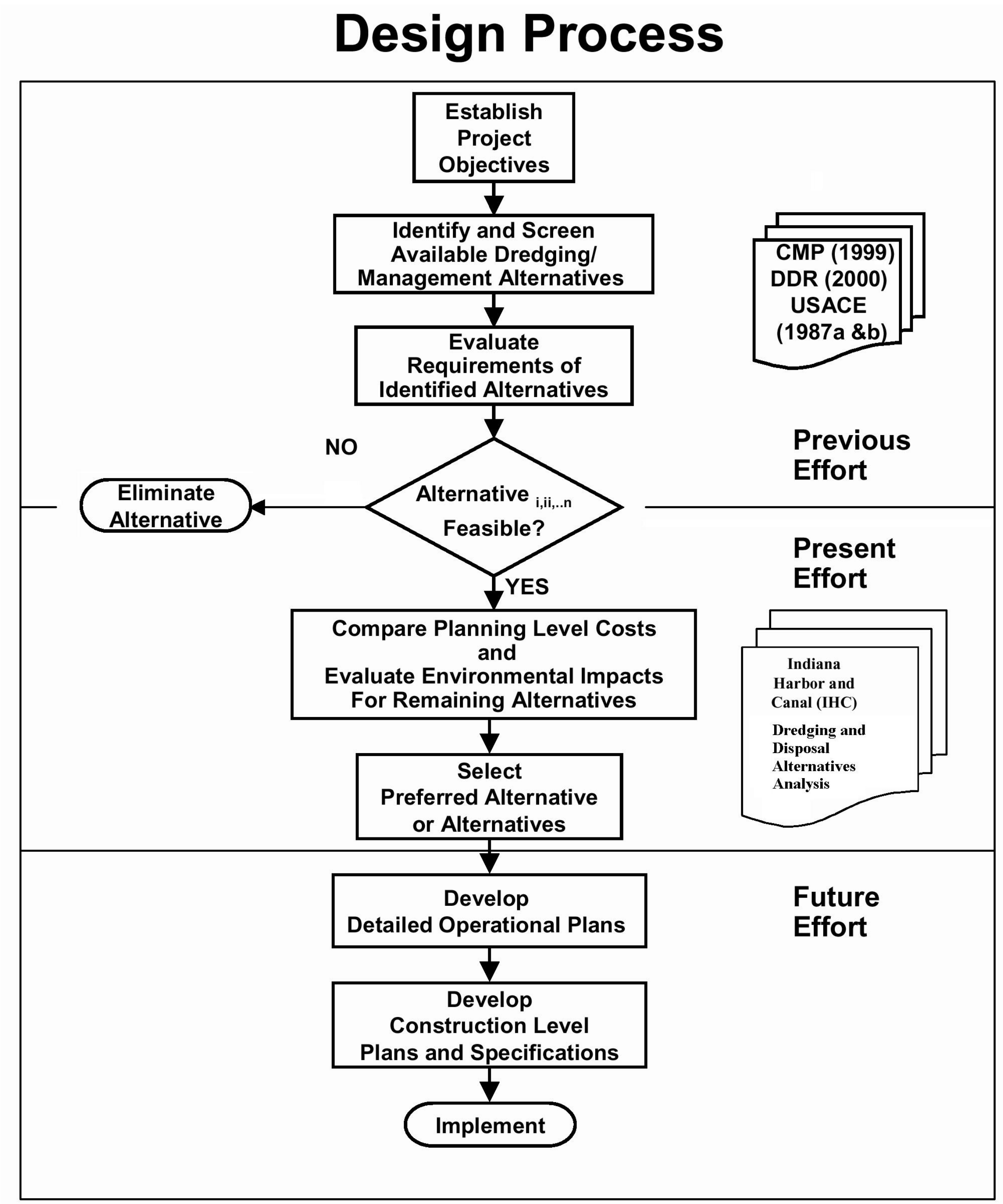

Figure 2. Overall design process

Project Objectives

This study was conducted in response to the public's desire to re-evaluate the use of hydraulic dredging and disposal for this project. The primary objective of the present study was to perform a comparative, planning-level evaluation of a limited number of dredging and placement alternatives for the operation of the Indiana Harbor and Canal Confined Disposal Facility (CDF). Specific objectives of the study were as follows:

- Compatibility of hydraulic dredging and placement with the existing proposed CDF design, developed for mechanical dredging and documented in the Design Documentation Report (DDR) (USACE, Chicago 2000)
- Feasibility of expediting backlog dredging using hydraulic dredging rather than mechanical dredging
- Relative air emissions (volatile and particulate) from the CDF for hydraulic dredging versus mechanical dredging
- Overall life cycle cost of hydraulic dredging versus mechanical dredging

Approach

The following four dredging and rehandling alternatives were considered in the alternatives analysis:

- Alternative 1 - Mechanical dredging and placement, as described in the DDR (USACE, Chicago 2000)
- Alternative 2 - Hydraulic dredging and placement at a baseline annual rate comparable to Alternative 1 (mechanical dredging)
- Alternative 3 - Hydraulic dredging and placement at an accelerated annual rate that would remove the backlog in a compressed timeframe (less than the 10 years specified in the DDR)
- Alternative 4 – Hydraulic dredging and placement at an accelerated annual rate that would remove the backlog in a compressed timeframe, with mechanical dredging of TSCA sediments

In order to provide a technical, environmental and economic basis for comparison of the alternatives, a planning-level evaluation was performed using available models and methodologies with existing information obtained from the previously referenced reports and available historical data. Specifically, the following were considered in the alternatives evaluation:

- CDF height and area required to contain the dredged material for the proposed dredging methods and rates
- Maximum feasible expedited hydraulic dredging rate of backlog dredging
- Average and peak effluent flow rates and contaminant concentrations for each alternative

- Storage volume requirements for flow equalization and storm water management
- Estimated relative volatile and particulate emissions from the CDF for mechanical and hydraulic dredging
- Cost estimates

Alternative 4 was ruled out in an early stage of the evaluation, since mechanical offloading facilities would be required for just one year of the project, increasing the cost by the same factor as for Alternative 1, where the offloading facilities will be utilized for the life of the project.

2 - Alternatives Evaluation

Dredging Volumes and Rates

Dredging volumes and rates for the mechanical dredging alternative are described in the DDR, Appendix E, Table E-1 (USACE, Chicago 2000). Total backlog dredging volume is specified in Table E-1 as 2.292 million cubic yards, with completion in the 10th year of dredging. This volume reflects a total of approximately 1 million cubic yards backlog dredging in the federal channel, with additional dredging volumes from non-federal areas such as consent decree areas. Total project volume over the 31-year project period is specified as 4.829 million cubic yards. These volumes were used for all alternatives evaluated. Dredging rate for the accelerated hydraulic alternative (Alt. 3) was modified from that specified in Table E-1 to permit completion of the backlog dredging in 4 years and total project volume in 24 years. A portion, 146,000 cubic yards[1], of the backlog dredging volume is classified as sediment subject to disposal in accordance with the Toxic Substances Control Act (hereafter referred to as TSCA sediment or material). For all alternatives, it was assumed that the TSCA sediment would be dredged and placed in a single year: the 7th year of dredging for the mechanical dredging and baseline hydraulic dredging alternatives (Alt. 1 and 2) and the 4th year of dredging for the accelerated hydraulic alternative (Alt. 3).

Average daily production rates were based on an assumed dredge size and operating efficiency, which were selected by taking into account project depths, sediment character and volume, and logistical considerations related to the dredging operation, debris removal and offloading processes. For mechanical dredging, the DDR specifies the use of a clamshell dredge, but does not specify a size. For purposes of this study, a 10-cy bucket with a daily production rate of 4000 cubic yards, operating 16 hours per day, 6 days per week, was assumed. For hydraulic dredging, a 12-inch hydraulic dredge appears to be the smallest hydraulic dredge that will accommodate the required project depths. Larger dredges could be utilized, but because all effluent produced at the CDF must be treated in a wastewater treatment plant constructed for this purpose, minimization of the flow rate is desired to minimize capital costs for the treatment plant. For the purposes of this study, use of a 12-inch hydraulic dredge with two booster pumps was assumed. The dredge was assumed to be productive 14 hours per day, 6 days per week and to have an average hourly production rate of approximately 334 cubic yards, or 4700 cubic yards daily. Actual production will be somewhat less than this when booster pumps are required.

[1] 146,000 cubic yards is a conservative over-estimate of TSCA material, based on historical sediment data. Prior to finalizing the TSCA storage requirements for the CDF, a sediment survey will be completed to determine the actual volume of TSCA sediment.

The average opening date for the navigation season at the Straits of Mackinac (connecting Lakes Michigan and Huron) is March 21st; the average closing date is January 15th. Indiana Harbor is open for navigation throughout the year, except for occasional brief periods when drifting ice fields, driven by winds, jam the harbor entrance (USACE, Chicago 1999). For the purposes of this study, dredging is assumed to occur approximately between April 15 and October 15 annually.

Disposal

A confined disposal facility (CDF) is to be constructed on a brownfield site, known as the ECI Site, East Chicago, Indiana (USACE, Chicago 1999). This site consists of about 168 acres of land formerly occupied by an oil refinery owned by Atlantic Richfield Company and subsequently acquired by Energy Cooperative Industries (ECI). ECI filed for bankruptcy in 1981 and abandoned the site. In response to a bankruptcy court order, the old refinery, including oil tanks, pipelines, and buildings, was completely demolished above ground level. The site was leveled, cleaned of debris, covered with topsoil and seeded (USACE, Chicago 1999). The CDF would be constructed of earthen dikes, using off-site materials. The inside slope of the dike is to be lined with a 3-foot layer of compacted clay extending from the crest to the toe of the dike. To isolate the groundwater beneath the site, a soil bentonite slurry wall is to be constructed along the perimeter of the dike extending from the lower end of the clay liner to the clay strata 33 feet below the existing ground surface. After the CDF is filled, it is to be capped with 3’ clay, 0.5’ sand, 2’ clean fill, and 0.5’ topsoil and then seeded (USACE, Chicago 2000). Originally, a separate cell for TSCA material was to be constructed inside the facility on the south side. However, to allay concerns regarding nearby land uses this cell will be relocated to the north side of the facility. This assumption has been incorporated into the preliminary layout developed in the present study.

The exterior footprint of the CDF was assumed to encompass 140 acres, with the general configuration as indicated in Figure 3. Appendix A of the DDR describes the selected dike configuration. The dike cross-section shown in Appendix A, Plate A-6, of the DDR (USACE, Chicago 2000) specifies an exterior slope of 3 horizontal to 1 vertical and interior slope of 1 horizontal to 1 vertical as illustrated in Figure 4. A 25-foot wide crest is specified for access roads for the facility construction vehicles and operations. No specifications for interior dikes were provided; for the purposes of this study, interior dikes were assumed to have slopes of 1 horizontal to 1 vertical on both faces. Final dike heights differ between the alternatives and are a function of the rate of placement of the material and the rate of consolidation of the material between lifts. Available sediment storage area is calculated based on available area within the dikes at ½ the maximum sediment storage depth, as determined by consolidation analysis (Figure A1).

Placement lifts of dredged material (thickness at the end of the disposal period) for the hydraulic dredging alternatives were determined using the USACE ADDAMS

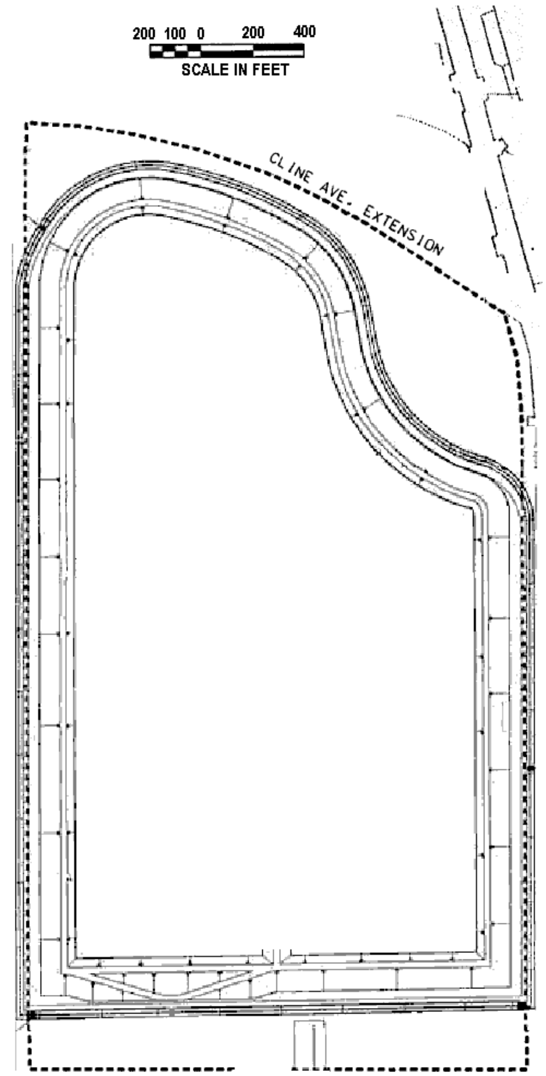

Figure 3. General configuration of CDF

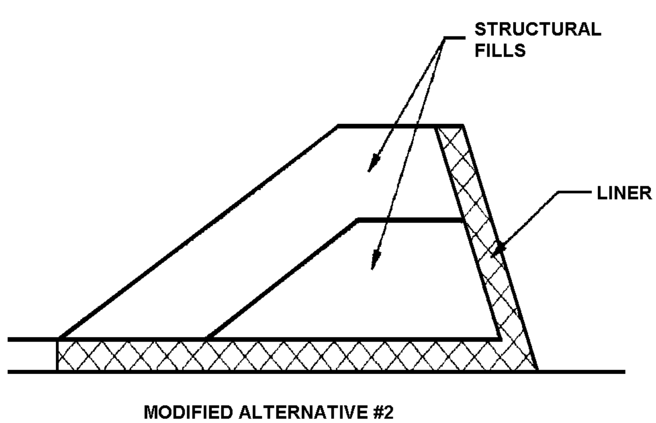

Figure 4. Selected perimeter dike configuration (USACE, Chicago 2000)

SETTLE module (Schroeder and Palermo 1995, Hayes and Schroeder 1992b), and settling data generated from settling tests run on IHC sediment (Environmental Laboratory 1987a and 1987b). Lifts for the mechanical dredging alternative were determined using an assumed bulking factor of 1.1 (USACE 1987). Rate of consolidation was evaluated using the USACE ADDAMS Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill (PSDDF) model (Stark 1996) for the assumed lift heights and placement intervals. A more detailed description of these models and the supporting calculations is contained in Appendix A. From these analyses, maximum sediment heights over the project life were determined for the three alternatives. A customary freeboard depth of 2 feet was assumed. There was a further requirement that, during operation and placement of material in the CDF, the freeboard be sufficient to accommodate storm flows from a 100-year precipitation event (USACE, Chicago 2000). It was estimated in the DDR that this would require 75 acre-feet of storage. The selected freeboard depth is sufficient to accommodate this volume within the available storage area. Based on settling tests conducted with sediments from the IHC in 1986 (Environmental Laboratory 1987a and 1987b), an additional depth of 4 feet for ponding is also required to facilitate adequate primary settling for hydraulic dredging alternatives. Maximum required dike height was then determined for each alternative by summing the maximum sediment height plus freeboard and ponding requirements. Construction staging, initial and final dike heights, and material placement intervals are more fully described for each alternative in the following CDF design summary.

For hydraulic dredging alternatives, effluent SS from primary settling was calculated based on the USACE ADDAMS DYECON module (Hayes and Schroeder 1992a) and the USACE ADDAMS SETTLE module (Hayes and Schroeder 1992b) using settling data from the 1986 settling tests (Environmental Laboratory 1987a and 1987b). Additional discussion regarding estimation of effluent SS can be found in Appendix A. For mechanical dredging and for effluent from secondary settling basins for hydraulic dredging, estimated effluent SS are based on field observations from other projects and professional judgment.

CDF Design Summary

CDF Cell Configuration

For all alternatives, a maximum footprint for the CDF of 140 acres was assumed. A CDF configuration of 5 cells was assumed for all dredging alternatives, with the smallest cell serving as an equalization basin. The approximate layout for Alternative 1 (mechanical dredging) is shown in Figure 5. In mechanical dredging, effluent flow rates are very low and the effluent contains low concentrations of suspended solids. As a consequence, chemical clarification is not required to remove solids. Because all water leaving the site must pass through the wastewater treatment plant (WWTP), however, some storage capacity must be provided to temporarily contain flows from major storm events. This will allow the WWTP capacity to be minimized. The fifth cell serves the function of an equalization basin for the mechanical dredging alternative. All dikes will be constructed from off-site materials. Cell 1, the TSCA cell, and the equalization basin will be constructed prior to the first year of dredging. Cells 2 and 3 will be constructed the following year. The dikes for all cells will initially be constructed to a height of 15.5 ft, approximately that specified in the DDR (USACE, Chicago 2000). This is sufficient to accommodate all of the backlog dredging. After completion of the backlog dredging, the dikes will be raised to their full and final height of 30 ft. After the dikes have been raised, dredged material can be placed in the equalization cell to more fully use the storage capacity of the site, but a specified minimum capacity must be maintained in the cell to provide for storm flow equalization until the primary cells are filled and capped. At that time, the remaining capacity of the equalization basin could be used for placement of additional mechanically dredged sediments. (Storm flow equalization is more fully discussed in the WWTP Flows section.)

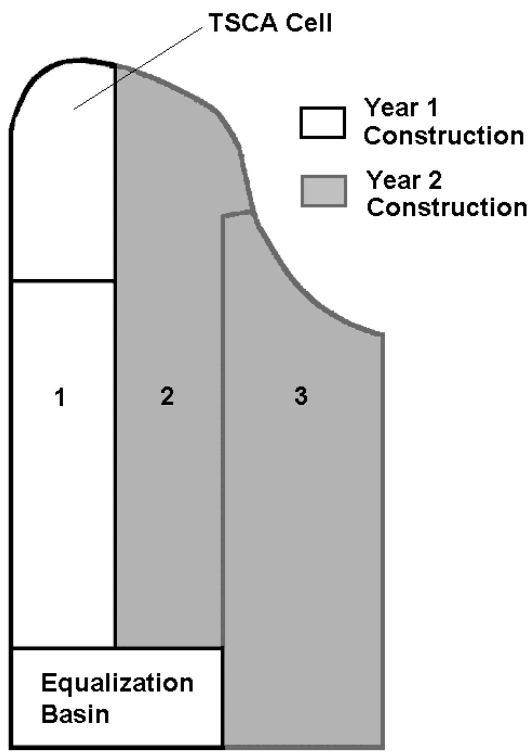

Figure 5. Approximate CDF configuration for alternative 1, mechanical dredging

For the hydraulic dredging alternatives, secondary settling enhanced by application of flocculants is required to reduce the suspended solids in the effluent prior to entering the WWTP. Capacity of the WWTP will be sufficient to handle sustained high flow rates, as necessitated by hydraulic dredge discharge flows, and separate storm flow capacity will not be required. Standby capacity of the WWTP could be reduced, however, if separate storm flow storage was provided. This is more fully discussed under WWTP Flows. A small equalization basin will be constructed and available to provide flow equalization at all times. Alternatively, the TSCA cell may be used as an equalization basin when not in use for material placement, or for emergency storm water retention, providing additional hydraulic retention time and further reducing suspended solids loading to the WWTP. The principal disadvantage to this is that the dewatering of material placed in the TSCA cell will be delayed. The approximate layout is illustrated in Figure 6. All dikes will be constructed of off-site materials. All cells may be constructed in one year to an initial lift height sufficient to accommodate the backlog dredging (all cells will be constructed to the same height, 21 ft for alternative 2, 24.5 ft for alternative 3). Alternatively, for alternative 2, standard hydraulic dredging, construction could be performed in two years. Cell 2 and the TSCA cell would be constructed prior to the first year of dredging and Cells 1 and 3 and the 2-acre equalization basin constructed the following year (Figure 6). This would require that the order of material placement be reversed for Cells 1 and 2 for the first 6 years of dredging. This would have minimal affect on material depth in both cells. This was assumed for cost estimating. For alternative 3, accelerated hydraulic dredging, the only construction that could be deferred to year 2 is Cell 3. This was assumed for cost estimating. For both hydraulic dredging alternatives, after backlog dredging is completed, dikes will be raised to their full and final height (32.5 ft for alternative 2; 33 ft for alternative 3).

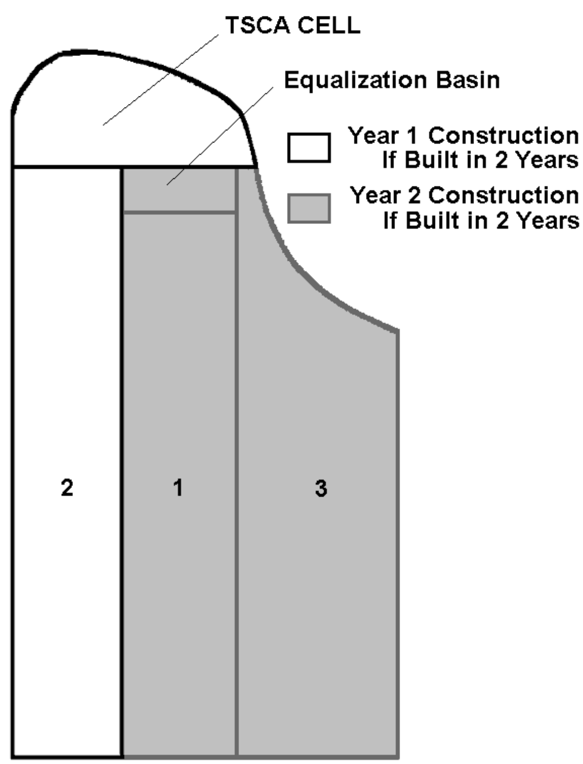

Figure 6. Approximate CDF configuration for alternatives 2 and 3, hydraulic dredging

Placement of TSCA Material

For all alternatives, non-TSCA sediment will be placed in the TSCA cell before and after placement of the TSCA material, providing an underlayer and overlayer of at least 3 feet of non-TSCA material after dewatering and consolidation.

Dredging and Storage Volumes

General dike configuration was discussed in previous sections. The selected dike configuration assumes that dikes are built outward as they are raised, rather than bearing on the previously placed dredged material. During construction of the initial dike lift, the dikes must therefore be inset from the site perimeter sufficiently to accommodate the full dike width within the specified footprint when the dikes are raised to their maximum height. The general dike height needs are summarized in Table 1. Annual production and cell placement (in order of use) are summarized in Table 2. Annual production for alternatives 1 and 2 were based on Table E-1 of the DDR. Annual production for alternative 3 was accelerated for the backlog dredging. Annual volumes for the subsequent maintenance dredging were modified slightly from those specified for Alternatives 1 and 2, to accommodate the equal size primary cells necessitated by the accelerated backlog dredging phase. Average annual number of dredging days for alternatives 1, 2, and 3 are 55, 48, and 42 (179 during accelerated backlog dredging) respectively, excluding non-dredging years. During years in which material would be placed in multiple cells, it is assumed that the specified volumes for each cell would be placed sequentially (first one cell, and then the other). If the TSCA cell is to be utilized for equalization during years in which dredged material is also to be placed there, this could be accomplished by first placing material specified for disposal in the TSCA cell, permitting sedimentation to take place, and then placing material specified for other cells. Effluent could then be received in the TSCA cell from the primary cells, although this would slow dewatering of the material placed in the TSCA cell. Additional information pertaining to each alternative follows.

Table 1. Average Dredged Material Storage Requirements

Alternative	Maximum [1] Dredged Material Elevation from Backlog Dredging (ft)	Initial Required Dike Height (ft)	Maximum [2] Dredged Material Elevation over Project Life (ft)	Final Required Dike Height (ft)	Consolidated Dredged Material Surface Elevation at 30 years (ft)
Alt 1	13.3	15.5	26.5	28.5	25.4
Alt 2	14.7	21.0	26.5	32.5	24.3
Alt 3	18.2	24.5	27.1	33	24.1

[1] Initial dike height was determined using this value, plus freeboard and ponding depths.
[2] Final dike height was determined using this value, plus freeboard and ponding depths.

Table 2. Dredged Material Placement Schedule

Year	Alternative 1 Mechanical: Volume (cy)	Alternative 1 Mechanical: Lift Thickness (ft)	Alternative 1 Mechanical: Cell No.	Alternative 2 Standard Hydraulic: Volume (cy)	Alternative 2 Standard Hydraulic: Lift Thickness (ft)	Alternative 2 Standard Hydraulic: Cell No.	Alternative 3 Accelerated Hydraulic: Volume (cy)	Alternative 3 Accelerated Hydraulic: Lift Thickness (ft)	Alternative 3 Accelerated Hydraulic: Cell No.
1	85333 42667	3.5 2.4	1 TSCA	128000	4.4	1	357667 357667	10.2 10.2	1 2
2	132017 162983	3.1 3.1	2 3	132017 122983 40000	4.5 3.4 4.3	2 3 TSCA	322667 322667 70000	8.7 9.3 7.7	3 1 TSCA
3	89333 44667	3.6 2.5	1 TSCA	134000	4.6	1	357667 357667	10.2 9.8	2 3
4	136939 169061	3.2 3.2	2 3	136939 119061 50000	4.7 3.3 5.2	2 3 TSCA	146000 *	13.6	TSCA
5	140000	5.7	1	140000	4.8	1	0	Dikes Raised	
6	142309 175691	3.4 3.4	2 3	142309 175691	4.8 4.7	2 3	152220 50740	5.1 5.3	1 TSCA
7	146000 *	8.3	TSCA	146000 *	13.6	TSCA	152220	4.7	2
8	147680 182320	3.5 3.5	2 3	147680 182320	5.0 4.9	2 3	152220	5.3	3
9	102000 51000	4.2 2.9	1 TSCA	103000 50000	3.9 5.2	1 TSCA	0	Dewatering	
10	153050 188950	3.6 3.6	2 3	153050 188950	5.2 5.0	2 3	152220 50740	5.1 5.3	1 TSCA
11	0	Dikes Raised		0	Dikes Raised		152220	4.7	2
12	106000 53000	4.3 3.0	1 TSCA	159000	5.7	1	152220	5.3	3
13	159000	3.8	2	159000	5.4	2	0	Dewatering	
14	196000	3.8	3	156000 40000	4.2 4.3	3 TSCA	152220 50740	5.1 5.3	1 TSCA
15	0	Dewatering		0	Dewatering		152220	4.7	2
16	106667 53333	4.4 3.0	1 TSCA	160000	5.8	1	152220	5.3	3
17	160000	3.8	2	120000 40000	4.1 4.3	2 TSCA	0	Dewatering	
18	194000	3.7	3	194000	5.2	3	152220 50740	5.1 5.3	1 TSCA
19	0	Dewatering		0	Dewatering		152220	4.7	2
20	76000 76000	3.1 4.3	1 TSCA	152000	5.5	1	152220	5.3	3
21	152000	3.6	2	112000 40000	3.9 4.3	2 TSCA	0	Dewatering	
22	193000	3.7	3	193000	5.1	3	152220 50740	5.1 5.3	1 TSCA
23	0	Dewatering		0	Dewatering		152220	4.7	2
24	77000 77000	3.1 4.4	1 TSCA	154000	5.6	1	152220	5.3	3
25	154000	3.7	2	154000	5.2	2			
26	195000	3.8	3	150000 45000	4.1 4.8	3 TSCA			
27	0	Dewatering		0	Dewatering				
28	78000 78000	3.2 4.4	1 TSCA	156000	5.6	1			
29	156000	3.7	2	111000 45000	3.9 4.8	2 TSCA			
30	198000	3.8	3	198000	5.2	3			

* TSCA Material. Other placements in TSCA cell are non-TSCA material.

Alternative 1 (Mechanical Dredging as described in DDR). Alternative 1 assumes 3 primary cells, one with an average storage area of 16.7 acres (Cell 1), one with approximately 28.7 acres (Cell 2) and one with an average storage area of approximately 35.4 acres (Cell 3). A fourth cell will be constructed for TSCA material, and will have an average storage area of 12 acres. A 10-acre fifth cell will provide storm flow equalization. Completion of backlog dredging in ten years was assumed, with the TSCA material being dredged in the 7th year, and no dredging occurring in the 11th year while the dikes are being raised. Maintenance dredging would then be completed over the remaining 19 years, with disposal cycling through each of the three primary cells and the TSCA cell, as summarized in Table 2. As previously mentioned, the equalization basin could be used to provide additional storage capacity of mechanically dredged sediments in later years. A functional ponding depth of 14.2 ft, exclusive of freeboard, is required to contain the maximum predicted storm flow storage requirement of 142-acre ft. However, a functional ponding depth of 13.5 feet will allow 2 ft of freeboard within the initial specified dike height of 15.5 feet and will provide storage adequate for over 99% of the predicted storm flows. Storm flows exceeding this can be retained in the primary cells temporarily, until the equalization basin has been pumped down sufficiently to accommodate the excess volume. Availability of the minimum 135-acre ft storage volume must be maintained until closure and capping of one or more of the primary cells. At that time, runoff from storm flows would be expected to be reduced proportionately to the reduction in contributing area, assuming storm flow from capped areas did not need to be captured and treated. This is more fully explained in the WWTP section. No allowance was made for this additional storage capacity in determining placement of the stated project volumes, however. The initial required dike height is 15.5 ft and the final required dike height is 28.5 ft. However, dikes will be constructed to a final height of 30 ft, consistent with the design documented in the DDR.

Alternative 2 (Hydraulic dredging comparable to Alternative 1). Alternative 2 assumes 3 primary cells, two with average storage areas of approximately 27.6 acres, one with an approximate storage area of 34.1 acres and an equalization/TSCA cell with an available storage area of 10 acres. Completion of backlog dredging in ten years was assumed, with the TSCA material being dredged in the 7th year, and no dredging occurring in the 11th year while the dikes are being raised. Maintenance dredging would then be completed over the remaining 19 years, with disposal cycling through each of the three primary cells and the TSCA cell as summarized in Table 2. A 2-acre equalization basin will be constructed in Cell 1 during initial construction and will serve to equalize flows to the WWTP and provide for secondary settling and SS removal. The TSCA cell may also serve as an equalization/secondary settling basin for the primary cells when not being utilized for disposal. The additional retention time would enhance SS removal, and settled solids would further encapsulate the TSCA material placed there. However, dewatering and consolidation of materials previously placed in the TSCA cell may be somewhat retarded as a result. During years in which the TSCA cell is scheduled to receive material, equalization of effluent from the primary cells could still take place in this cell if needed, if material was first placed in the TSCA cell and allowed to undergo sufficient preliminary settling. Depending upon the rate of solids carryover into the secondary settling basin, some solids removal could be necessary, but projected solids accumulations are low and no need for solids removal is expected. The initial required dike height is 21 ft; the final required dike height is 32.5 ft.

Alternative 3 (Hydraulic dredging accelerated timeframe). Alternative 3 assumes 3 primary cells with average storage areas of approximately 29.8 acres and an equalization/TSCA cell with an available storage area of 10 acres. Completion of backlog dredging in four years was assumed, with the TSCA material being dredged in the 4th year, and no dredging occurring in the 5th year while the dikes are being raised. Maintenance dredging would then be completed over the remaining 19 years, with disposal cycling through each of the three primary cells and the TSCA cell as summarized in Table 2. As in Alt 2, a 2-acre equalization basin will be constructed in Cell 1 during initial construction to equalize flows and to provide for secondary settling and SS removal. As previously stated, the TSCA cell could also be utilized intermittently, as needed. Projected solids accumulations in the secondary settling basin are considered to be comparable to Alt 2. The initial required dike height for Alt 3 is 24.5 ft and the final required dike height is 33 ft.

Pumping

Pumping will be required to transfer water from the disposal cells to the equalization basin and from the equalization basin to the WWTP. Pumping will also be required to maintain the groundwater gradient specified under the RCRA requirements for the ECI site. Pumps should be sized for the maximum anticipated flows, but typical operational costs are based on mean anticipated flows. Combined flows resulting from hydraulic dredge discharge, dredged material consolidation, precipitation, and groundwater control systems are described in the WWTP Flows section (Table 6). Table 3 summarizes the average annualized estimated pumping requirements for dredging and non-dredging periods throughout the entire project. In Table 3, transfer pumps are larger capacity pumps designed to handle dredge discharge flows (transfer pumps). Standpipe pumps are smaller capacity pumps designed to handle storm flows during non-dredging periods, which may also provide excess capacity if needed to handle peak flows during dredging, or in the event of the failure of another pump.

For mechanical dredging, pumpout capacity from the cells to the equalization basin will be dictated by storm flows. Assuming a major storm event with a 1-year return period, a 300-gpm pumping rate from each cell simultaneously would be required to empty the primary cells into the equalization basin over a period of 4 or 5 days. Longer or shorter times could be assumed and pumping capacity adjusted accordingly. However, given the infrequency of the event, designing for higher flows may not be justifiable from a cost perspective. Pumping rate from the equalization basin to the treatment plant will be determined by plant capacity. For the mechanical dredging alternative, 200 gpm was the assumed plant capacity. (The basis for plant capacity determinations is more fully described under WWTP Flows.)

Table 3. Annualized Estimated Pumping Requirements

Alternative	Transfer Pumps during Dredging		Standpipe Pumps (non-dredging)		Equalization to Treatment Pumps	
	Capacity	Actual	Capacity	Actual	Capacity	Actual
Alternative 1 (Mechanical) 32 years [1]	none	none	2 @ 300 gpm (Cells 2 & 3) 1 @ 200 gpm (Cell 1) 1 @ 100 gpm (TSCA Cell)	14,000,000 gal/yr each 33 days @ 300 gpm each 9,300,000 gal/yr 33 days @ 200 gpm 4,700,000 gal/yr 33 days @ 100 gpm	1 @ 200 gpm 365 days/yr [2]	42,000,000 gal/yr 146 days @ 200 gpm
Alternative 2 (Hydraulic) 32 years [1]	1 @ 3100 gpm 38 days/yr [2] (average annual dredging duration [3])	120,000,000 gal/yr 27 days @ 3100 gpm or 38 days @ 2200 gpm	3 @ 300 gpm (Cells 1, 2 & 3) 1 @ 100 gpm (TSCA Cell)	14,000,000 gal/yr each 33 days @ 300 gpm each 4,700,000 gal/yr 33 days @ 100 gpm	1 @ 2700 gpm 38 days/yr [2] 1 @ 200 gpm 365 days/yr [2]	120,000,000 gal/yr 31 days @ 2700 gpm or 38 days @ 2200 gpm 47,000,000 gal/yr 163 days @ 200 gpm

[1] Number of years pumping would be required, including non-dredging years, and 2 years post-dredging, prior to capping.
[2] Period of time pump must be available on-site (equivalent to total dredging duration for dredging transfer pumps when booster pumps are not required).
[3] Including non-dredging years and 2 years post-dredging, prior to capping.
[4] Pumping requirements during accelerated backlog dredging.
[5] Pumping requirements during TSCA & maintenance dredging, including non-dredging years, and 2 years post-dredging, prior to capping.
NOTE: The number of days of pumping was used for O&M cost calculations. The number of days is not meant to imply that the operation of the CDF and WWTP would be discontinuous.

(Continued)

Table 3. Annualized Estimated Pumping Requirements

Alternative	Transfer Pumps during Dredging		Standpipe Pumps (non-dredging)		Equalization to Treatment Pumps	
	Capacity	Actual	Capacity	Actual	Capacity	Actual
Alternative 3 (Accelerated Hydraulic) 3 years [4]	1 @ 3100 gpm 179 days/ yr [2] (average annual dredging duration)	598,000,000 gal/yr 134 days @ 3100 gpm 179 days @ 2300 gpm	3 @ 300 gpm (Cells 1, 2 & 3) 1 @ 100 gpm (TSCA Cell)	15,000,000 gal/yr each 35 days @ 300 gpm each 4,700,000 gal/yr 33 days @ 100 gpm	1 @ 2700 gpm 179 days/yr [2] 1 @ 200 gpm 365 days/yr [2]	598,000,000 gal/yr 154 days @ 2700 gpm or 179 days @ 2300 gpm 50,000,000 gal/yr 174 days @ 200 gpm
Alternative 3 (Accelerated Hydraulic) 23 years [5]	1 @ 3100 gpm 30 days/ yr [2] (average annual dredging duration [3])	89,000,000 gal/yr 20 days @ 3100 gpm 30 days @ 2100 gpm	3 @ 300 gpm (Cells 1, 2 & 3) 1 @ 100 gpm (TSCA Cell)	14,000,000 gal/yr each 33 days @ 300 gpm each 4,700,000 gal/yr 33 days @ 100 gpm	1 @ 2700 gpm 30 days/yr [2] 1 @ 200 gpm 365 days/yr [2]	89,000,000 gal/yr 23 days @ 2700 gpm or 30 days @ 2100 gpm 47,000,000 gal/yr 163 days @ 200 gpm

[1] Number of years pumping would be required, including non-dredging years, and 2 years post-dredging, prior to capping.
[2] Period of time pump must be available on-site (equivalent to total dredging duration for dredging transfer pumps when booster pumps are not required).
[3] Including non-dredging years and 2 years post-dredging, prior to capping.
[4] Pumping requirements during accelerated backlog dredging.
[5] Pumping requirements during TSCA & maintenance dredging, including non-dredging years, and 2 years post-dredging, prior to capping.
NOTE: The number of days of pumping was used for O&M cost calculations. The number of days is not meant to imply that the operation of the CDF and WWTP would be discontinuous.

(Concluded)

For the hydraulic alternatives, pumpout capacity from the cells to the equalization basin will be dictated by the peak dredge discharge, approximately 5300 gpm. However, assuming the dredge operates 14 hours per day and the cell is pumped 24 hours per day, pump capacity could be reduced. A capacity of 3100 gpm would be sufficient to transfer the daily production in a 24-hour period throughout the dredging period. (In order to allow for pump down time or draindown of precipitation also, in both the hydraulic and mechanical dredging alternatives, some excess pump capacity may be advisable; 100-300 gpm excess capacity was assumed, depending upon the size of the cell, which could be provided by the standpipe pumps specified in Table 3.) Pumpout from the equalization basin to the treatment plant is again determined by plant capacity, 2700 gpm (see WWTP Flows). During hydraulic disposal, only the active cell would require pumping, assuming no precipitation. During major storm events, all cells would be pumped simultaneously for both mechanical and hydraulic alternatives.

Capping

The dredged material must be sufficiently consolidated to support an overburden before a cap can be placed. Based on the consolidation analysis and the depths specified herein, for all alternatives (mechanical and hydraulic), the material is expected to reach its desiccation limit within two years of placement of the final lift. At that time, the material could be capped. Capping could be staged, as each cell is filled and consolidated, or completed all at one time, after the final cell is filled and consolidated. Years of final placements are given in Table 2. At the time cells are completely filled, TSCA material will be overlain by non-TSCA material, so staging of capping would appear to be largely an administrative decision and regulatory decision. Capping 2 years following the final placement of material in the CDF was assumed for costing purposes.

Effluent Suspended Solids

Each stage of the CDF and the WWTP will produce effluent that is different from the previous stage with respect to suspended solids and/or contaminant concentrations. It is the function of the CDF to reduce SS, prior to release of effluent from the CDF to the WWTP. It is the function of the WWTP to remove dissolved contaminants and remaining suspended solids, if necessary, prior to release of effluent from the WWTP to the adjacent waterway.

Effluent leaving the primary disposal cells (primary effluent) will be highest in suspended solids. Secondary settling in the equalization basin, coupled with the use of flocculants, will produce a secondary effluent that is lower in SS than the primary effluent. Because many contaminants are associated primarily with particulate matter, total contaminant concentrations will also be lower for secondary effluent than for primary effluent. Dissolved contaminant concentrations will be largely unchanged between primary and secondary effluent, as settling removes only the particulate associated contaminants. The WWTP will contain treatment processes to reduce or remove the remaining dissolved and particulate associated contaminants. The following summarizes a comparison of estimated SS concentrations for the different alternatives. Estimated dissolved contaminant concentrations in the effluent are summarized under WWTP flows.

For hydraulic dredging, primary effluent SS concentrations are a function of the character of the slurry discharged by the dredge into the primary cells (grain size and percent solids, for example), the size and shape of the CDF, ponding depth and area, and the hydraulic retention time, which is a function of the rate of flow into the cell. Effluent SS from primary settling of the hydraulically dredged material is estimated based on the results of the USACE ADDAMS DYECON module (Hayes and Schroeder 1992a) and the USACE ADDAMS SETTLE module (Hayes and Schroeder 1992b) using settling data from the 1986 settling tests (Environmental Laboratory 1987a and 1987b). For mechanical dredging, effluent is primarily a result of precipitation and resulting surface runoff. Because settling tests have not been conducted on runoff from IHC sediments, estimated SS levels are based on project histories from other projects and laboratory settling tests on IHC sediment slurries. For both hydraulic and mechanical dredging, secondary effluent SS concentrations are based on field observations for similar projects and professional judgment (USACE 1987).

Alt 1 Mechanical Dredging. Effluent suspended solids concentrations from the primary cells are assumed to be low, comparable to that achieved with secondary settling for the hydraulic dredging alternatives (10 to 20 mg/l with the use of flocculants). However, runoff suspended solids concentrations from the primary cells may be much higher, depending on the degree of desiccation of the surface. During the early stages of drying, runoff suspended solids concentrations as high as 500 to 1000 mg/l are likely. During the latter stages of drying, the runoff suspended solids concentrations will be as low as 50 to 100 mg/l. After a couple days of detention in the equalization basin, the suspended solids concentration should decrease to 10 to 30 mg/l.

Alt 2 Standard Hydraulic Dredging. Effluent from the primary cells will be lower in SS than effluent from the TSCA cell, because the primary cells are larger, with a correspondingly higher hydraulic retention time (HRT) (assuming equivalent influent flow rates). HRT will vary, being greatest at the beginning of a disposal event when maximum ponding depth is available, and least at the end of a disposal event when only the minimum ponding depth is available. Effluent SS levels will be low initially, increasing as ponding depth diminishes. Maximum ponding depth was assumed to equal the minimum required ponding depth (4 ft) plus the anticipated lift thickness for the operational year in question (ranging from 3.3 to 5.8 ft for the primary cells, and 4.3 to 13.6 ft for the TSCA cell), less a nominal 1 ft allowance for accumulation of solids occurring during the initial flooding of the cell to the maximum ponding depth. Minimum SS levels for primary cells were estimated to range from <1 to 5 mg/l, for HRTs ranging from 244 to 301 hours, at initial ponding depths of approximately 7.5 to 8 ft. Maximum SS levels for the primary cells were estimated to range from 60 mg/l to 180 mg/l total suspended solids (TSS), for HRTs ranging from 120 to 160 hours at the minimum ponding depth of 4 ft. Minimum SS from the TSCA cell were estimated at

1150 mg/l, for a HRT of 60 hours, and initial ponding depth of 7.6 ft; maximum SS concentrations were estimated at 3040 mg/l TSS, for a HRT of 31 hours, and a ponding depth of 4 ft. SS levels must be reduced prior to the effluent entering the WWTP, or unit operations must be incorporated in the WWTP to remove the SS. The equalization basin is intended to facilitate removal of SS prior to the WWTP. Flocculants may be added to the primary effluent to improve removal of SS in the equalization basin. There is a relationship between SS loads in the primary effluent and SS concentrations in the secondary effluent. For an influent SS to the equalization basin of up to 180 mg/l (primary cell effluent SS), secondary effluent SS are assumed to range from 10 to 20 mg/l with the use of flocculants. For an influent SS of 3300 mg/l (TSCA effluent SS), secondary effluent SS are assumed to range from 50 to 80 mg/l TSS, with the use of flocculants.

Runoff SS from the primary cells may be much higher than effluent SS, depending on the degree of desiccation of the surface. During the early stages of drying, runoff SS concentrations as high as 5000 to 10000 mg/l are likely. During the latter stages of drying, the runoff SS concentrations will be as low as 100 to 200 mg/l. Flocculants may therefore be required during some periods to lower the SS concentrations in runoff as well. Alternatively, during the off season when SS concentrations in the runoff are expected to be lower, the runoff could be detained in the primary cells and equalization basin until the suspended solids concentration decreases to the required level. A detention of 7 to 10 days should be sufficient to reduce concentration to 10 to 30 mg/l.

Alt 3 Accelerated Hydraulic Dredging. As for Alt 2, HRT will vary. Effluent SS levels will be low at the beginning of a disposal event and will increase to maximum values as ponding depth approaches its minimum. Maximum ponding depth was assumed to equal the minimum required ponding depth (4 ft) plus the anticipated lift thickness for the operational year in question (ranging from 4.7 to 10.2 ft for the primary cells, and 5.3 to 13.6 ft for the TSCA cell), less a nominal 1 ft allowance for accumulation of solids occurring during the initial flooding of the cell to the maximum ponding depth. Minimum effluent SS were estimated to be <3 mg/l during both the accelerated dredging phase and the maintenance dredging phase, for HRT ranging from 265 to 444 hours, and initial ponding depths of 12.3-12.8 ft during accelerated dredging, and 7.8 to 8.3 ft during maintenance dredging. Maximum effluent SS from the primary settling basins, discharging to the secondary settling basin, were estimated at 100 to 140 mg/l TSS, for a HRT ranging from 129 to 139 hours, and ponding depth of 4 ft. Minimum effluent SS from the TSCA cell was estimated to be 80 mg/l, for a HRT of 148 hours and initial ponding depth of 16.1 ft, during the year of TSCA dredging only. Minimum effluent SS from the TSCA cell during maintenance and accelerated dredging ranged from 198 to 956 mg/l for HRT of 117 and 65 hours, respectively. Maximum effluent SS from the TSCA cell were estimated to be 3040 mg/l TSS for a HRT of 31 hours and ponding depth of 4 ft.

Secondary effluent SS ranging from 10 to 20 mg/l is assumed to be achievable with the use of flocculants for a feed of 100 mg/l TSS and from 50 to 80 mg/l TSS for a feed of 3300 mg/l TSS. As for Alt 1 and Alt 2, runoff suspended solids concentrations from the primary cells may be much higher, depending on the degree of desiccation of the surface. Estimated concentrations due to runoff are considered to be comparable to the other two alternatives.

Consolidation Analysis

The rate of dredged material consolidation was evaluated for each alternative using the USACE PSDDF model (Stark 1996). Initial lift heights and void ratios were calculated using the USACE SETTLE model for the hydraulic dredging alternatives. Lift heights and initial void ratios for the mechanical dredging alternative were calculated based on the in situ void ratio and assuming a bulking factor of 1.1 at placement (Appendix A). The lift sequence and schedule for all alternatives and cells are given in Table 2.

The consolidation properties were generated from the geotechnical characterization tests conducted on IHC sediment in 1986. The void ratio-effective stress and void ratio-permeability relationships were developed by reducing the self-weight consolidation test data and then fitting a regression curve to the test data results to extend the data throughout the entire range of effective stresses and void ratios likely to be observed in the CDF. Figures 7 and 8 show the results of a regression analyses. The Atterberg limits were used to predict the dredged material saturation limit, desiccation limit, and desiccation crust thickness. Average monthly precipitation and potential evapotranspiration values were generated using the USEPA Hydrologic Evaluation of Landfill Performance (HELP) model (Schroeder et al. 1994). Good CDF design and management, promoting runoff and evapotranspiration, was assumed in the modeling. Figure 9 illustrates the average surface profiles for the three dredging alternatives based on the results obtained from PSDDF.

There are a number of references containing guidance on CDF management and passive dewatering methods. Placement of material in thin lifts will promote dewatering by reducing the drainage path. Compartmentalization of the CDF, as proposed here, will permit inactive cells to dry while disposal continues in active cells (USACE 1987, Palermo, Montgomery and Poindexter 1978). Good surface water management, minimizing the amount of time ponded water remains on the dredged material, is important to dewatering. Progressive trenching is the practice of induced surface drainage through construction of a network of trenches that are progressively deepened as dewatering progresses (Hayden 1978). Perimeter trenching is sometimes utilized, beginning with excavation of a trench along the inside perimeter of the CDF during construction. Consolidation of dredged material will be greater here, because less compressible foundation materials have been removed and replaced with dredged material. As a result, a natural trench will form along the perimeter as disposal progresses, further promoting dewatering of the interior of the CDF. Ponded water should be drained or pumped off as soon as possible (USACE 1987). Establishment of a suitable vegetative cover can promote dewatering through transpiration (Palermo, Montgomery and Poindexter 1978) and would have the additional benefit of reducing fugitive dust.

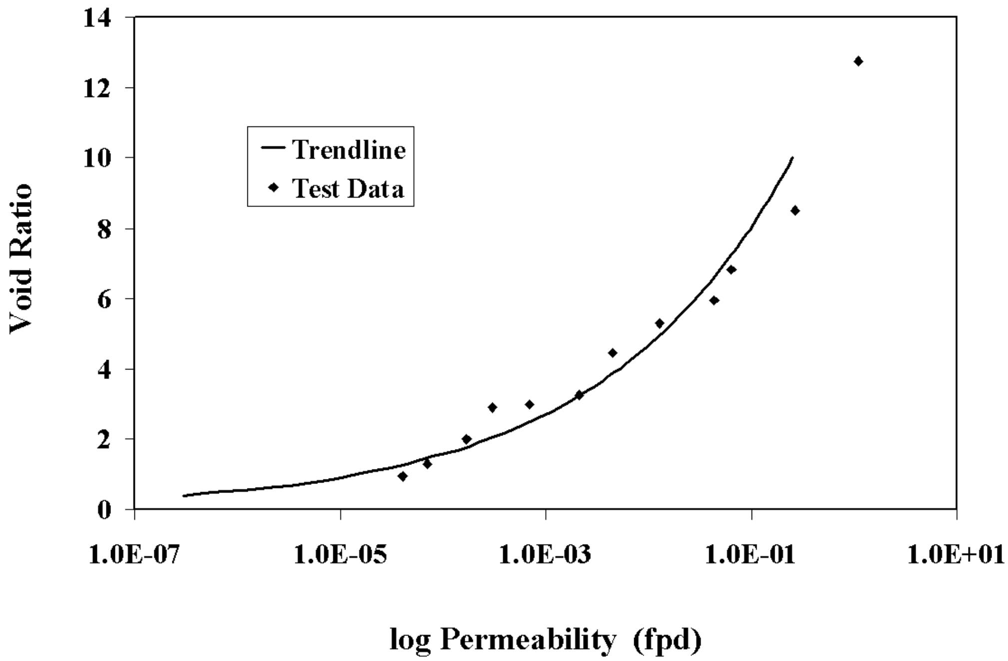

Figure 7. Void ratio versus log permeability

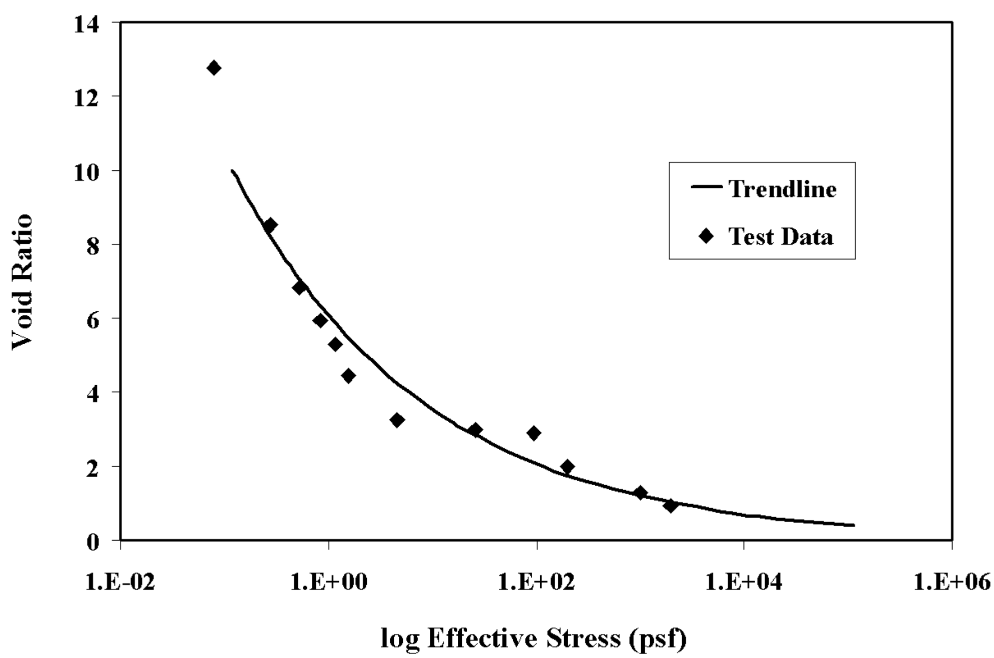

Figure 8. Void ratio versus log effective stress

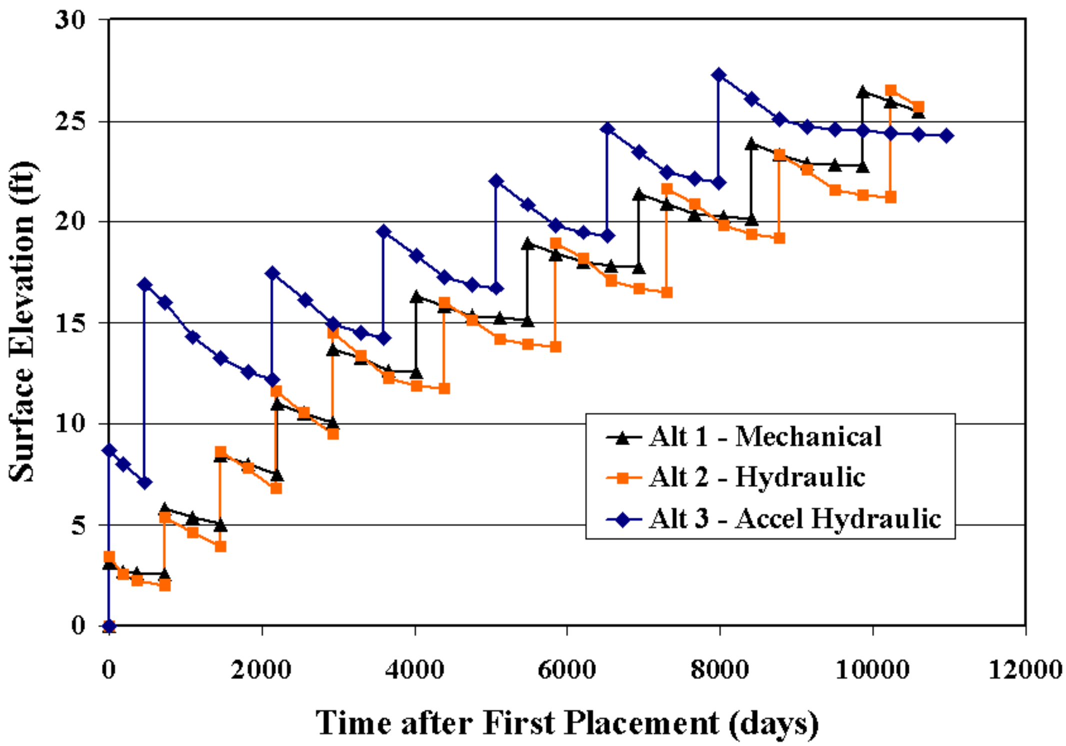

Figure 9. Average surface profiles for dredging alternatives

WWTP Flows

The Resource Conservation and Recovery Act (RCRA) closure and corrective action requirements for the ECI site require that a groundwater cutoff wall and gradient control system be operated at the site. Groundwater levels under the site must be maintained at least 2 feet below the prevailing level outside the cutoff wall. Flows resulting from the gradient control system (groundwater), disposal operations (effluent), precipitation (runoff) and consolidation of placed material (consolidation flows) will all be processed through a WWTP to ensure contaminant levels are below applicable water quality criteria prior to discharge to the IHC. As part of the present study, design flows for the WWTP were estimated based on anticipated dredging and consolidation rates, available historical sediment chemistry, and area precipitation data. Groundwater production during the initial drawdown period, discharge rates by the hydraulic dredge, and storm water flows are expected to govern WWTP capacity requirements.

The volume of groundwater that must be processed during the initial drawdown period was estimated to be approximately 42.4 million gallons (Appendix B, DDR). Constraints on the pumping rate utilized to establish the requisite drawdown are the infiltration rate through the slurry wall (estimated at 1 gpm, Appendix B, DDR), infiltration from precipitation, pumping capacity of the wells, and capacity of the WWTP. In this case, expected maximum capacity of the gradient control system is 1650 gpm (Appendix B, DDR), which was projected to permit completion of drawdown in approximately 20 days. For the same assumptions, assuming a 200-gpm WWTP capacity, drawdown was projected to require almost 5 months. The bulk of precipitation received during drawdown would be expected to infiltrate because drawdown will be accomplished while the CDF is empty, the upper soil layers are largely sandy in character, and drawdown is planned for the winter or early spring when little evaporation is expected. An additional analysis was conducted under this study to evaluate pumping requirements sufficient to offset expected infiltration during different drawdown periods and specific storage values. Net infiltration was estimated using precipitation and evaporation data generated using the HELP model. Tables 4 and 5 summarize the results of that analysis.

Since dredging would be expected to begin in mid-April, drawdown periods preceding mid-April and ranging from one to six months in duration were evaluated. Net infiltration averaged from approximately 1.61 to 9.94 inches for the periods evaluated. Pumping requirements for drawdown were also evaluated for drawdown periods ranging from 1 to 6 months, specific storage values of 0.2 and 0.3, and drawdown depths ranging from 2 to 4 feet, anticipating that additional drawdown might be desirable initially to offset infiltration associated with dewatering of the first lifts of dredged material. Pumping rates to achieve drawdown and accommodate net infiltration ranged from approximately 558 to 1389 gpm for a one-month period, from 402 to 679 gpm for a 3-month period, from 319 to 527 gpm for a 4-month period, and 212 to 350 gpm for a six-month period (Table 5 Upper 99% CL).

Table 4. Net Infiltration Estimates

Period	Net Infiltration			
	inches	feet	cfs	gpm
Mar				
Mean One-Month	1.61	0.134	0.104	47
95% One-Month[1]	3.83	0.319	0.246	110
99% One-Month[2]	4.93	0.411	0.317	142
Jan-Mar				
Mean Three-Month	3.75	0.312	0.241	108
95% Three-Month[1]	7.33	0.611	0.471	212
99% Three-Month[2]	9.12	0.760	0.587	263
Dec-Mar				
Mean Four-Month	5.78	0.482	0.279	125
95% Four-Month[1]	8.55	0.713	0.412	185
99% Four-Month [2]	9.94	0.828	0.479	215
Oct-Mar				
Mean Six-Month	6.06	0.505	0.195	87
95% Six-Month[1]	8.60	0.717	0.276	124
99% Six-Month[2]	9.87	0.822	0.317	142

[1] Mean plus 2 standard deviations
[2] Mean plus 3 standard deviations

Table 5. Drawdown Analysis							
Period	Duration (months)	Specific Storage	Depth (ft)	Drawdown	Drawdown Plus Net Infiltration		
				Pumping Rate (gpm)	Pumping Rate (gpm)		
					Mean	Maximum [1]	
						Upper 95% C. L.	Upper 99% C. L.
Mar	1	0.2	2	416	462	526	558
		0.2	3	623	670	734	766
		0.2	4	831	878	941	973
		0.3	2	623	670	734	766
		0.3	3	935	981	1045	1077
		0.3	4	1247	1293	1357	1389
Jan-Mar	3	0.2	2	139	247	350	402
		0.2	3	208	316	419	471
		0.2	4	277	385	489	540
		0.3	2	208	316	419	471
		0.3	3	312	420	523	575
		0.3	4	416	524	627	679
Dec-Mar	4	0.2	2	104	229	289	319
		0.2	3	156	281	341	371
		0.2	4	208	333	393	423
		0.3	2	156	281	341	371
		0.3	3	234	359	419	449
		0.3	4	312	437	497	527
Oct-Mar	6	0.2	2	69	157	193	212
		0.2	3	104	191	228	246
		0.2	4	139	226	263	281
		0.3	2	104	191	228	246
		0.3	3	156	243	280	298
		0.3	4	208	295	332	350

[1] Upper confidence limits based on potential range of infiltration

At peak flow, a 12-inch hydraulic dredge was assumed to produce effluent at a rate of 11.78 cfs or approximately 5300 gpm. The dredge is assumed to operate 14 hours per day, 6 days per week, yielding an effective flow rate after equalization of 5.89 cfs or approximately 2700 gpm. Combined with other surface and groundwater flows, the maximum peak flow rate is estimated to be 6.17 cfs or approximately 2800 gpm. This flow rate exceeds the highest rate estimated for drawdown requirements. Therefore, for the hydraulic dredging alternatives, the capacity of the WWTP is governed by the requirements of the dredge discharge during active disposal. During the off-season period, required treatment capacity will be generally lower, determined by storm, groundwater and consolidation flows. However, storm flows can be significant. For the mechanical dredging alternative, where effluent flows are expected to be much lower, treatment plant capacity could possibly be reduced if sufficient storage volume were available for equalization. Equalization basin requirements were therefore evaluated based on a reservoir analysis using 40 years of runoff predictions by the HELP model for consolidated dredged material with synthetic precipitation data. The maximum storage volume requirements for runoff assuming a constant discharge rate from the equalization basin were determined for different pumping rates (corresponding to WWTP inflow capacity). The results of this analysis indicate that WWTP capacity can be reduced to 200 gpm for the mechanical dredging alternative by providing a separate equalization basin in the CDF. A 10-acre basin providing 14 feet of ponded depth will meet the maximum estimated storm flow requirement of 142 acre-ft for the period evaluated. Additionally, only one six-month wet period during the 40 years of simulation required this much capacity; the remainder of the time the maximum storage capacity needed was about 80 acre-ft. For the 99th percentile of the data, approximately 92 acre-ft of storage would be required. For the 95th percentile of the data, approximately 51 acre-ft of storage would be required (Figure 10). A 10-acre basin with 13.5 feet of ponded depth (15.5 ft initial dike height, less 2 ft freeboard) would provide sufficient capacity for over 99% of the anticipated storm flows.

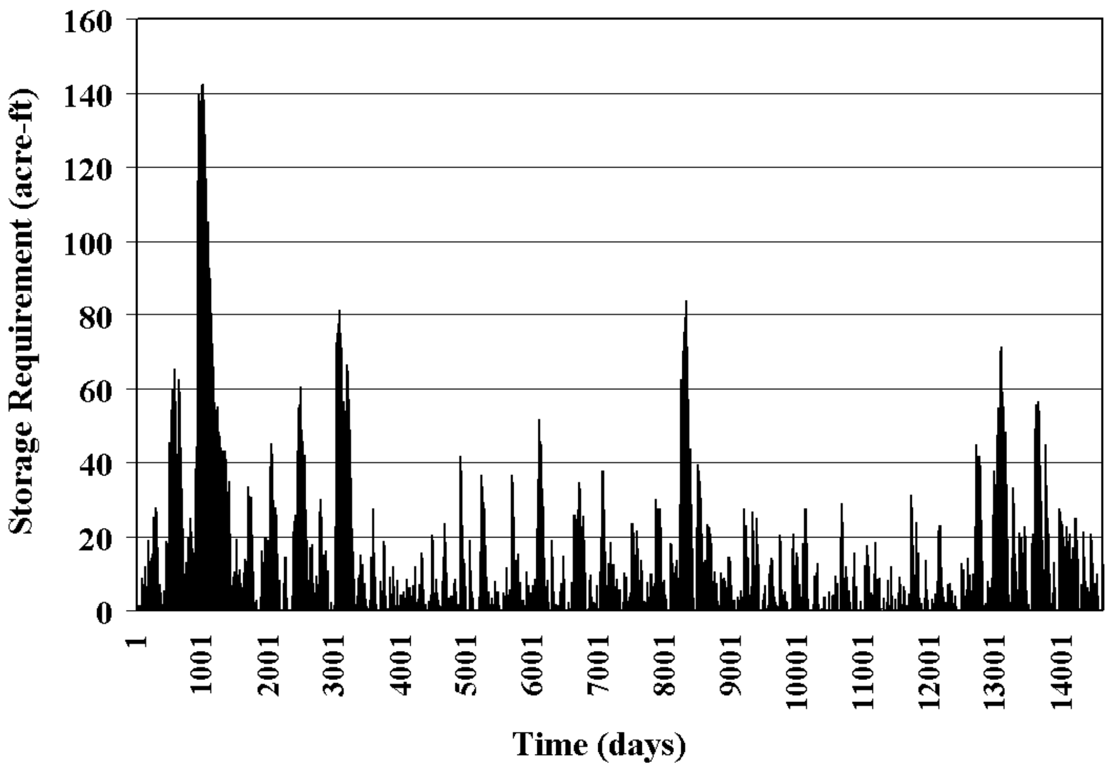

Figure 10. Runoff storage volume analysis for 10-acre equalization cell and a constant discharge rate of 200 gpm (mechanical dredging alternative)

In response to preliminary review comments, a similar analysis was conducted for the hydraulic dredging alternatives to evaluate the standby WWTP capacity required to handle storm flows. Storage requirements were evaluated for pumping rates ranging from 200 gpm to 2700 gpm, and equalization basins of 2 acres and 10 acres. Available ponding depths were determined based on the initial and final dike heights for the two hydraulic alternatives. At the maximum WWTP capacity of 2700 gpm, the 2-acre equalization basin specified in the CDF design section provides sufficient storage for the maximum anticipated storm flows, even at the lowest dike height of 21 feet (available storage volume of 38 acre-ft, Figure 11). At a standby capacity of 200 gpm, up to 143 acre-ft of storage would be required, which could be satisfied with an equalization basin of 10 acres, with 14.3 feet of storage depth, exclusive of freeboard. At 700 gpm, 49 acre-ft of storage is required (Figure 12). Figure 13 illustrates the frequency and variation in storage requirements based on the runoff analysis conducted, assuming a 10-acre equalization basin and 300-gpm pumpout rate. No storage volume exceedances are predicted for this basin size and standby plant capacity. Some intermediate equalization basin size and pumping rate could be selected, but there are diminishing returns for increasing pumping rates, as seen in Figures 11 and 12. If it is determined in the detailed design phase that cost savings could be achieved by reducing the standby capacity of the WWTP for the hydraulic dredging alternatives, a 3-acre equalization basin with 300 gpm pumping rate would provide adequate equalization for over 98% of the predicted storm flows. For those flows exceeding the storage capacity of the equalization basin, the primary cells could be utilized for temporary, short-term storage. This adjustment could be made to the CDF configuration specified herein (2-acre equalization basin) with minimal impact on lift depths and final dike heights.

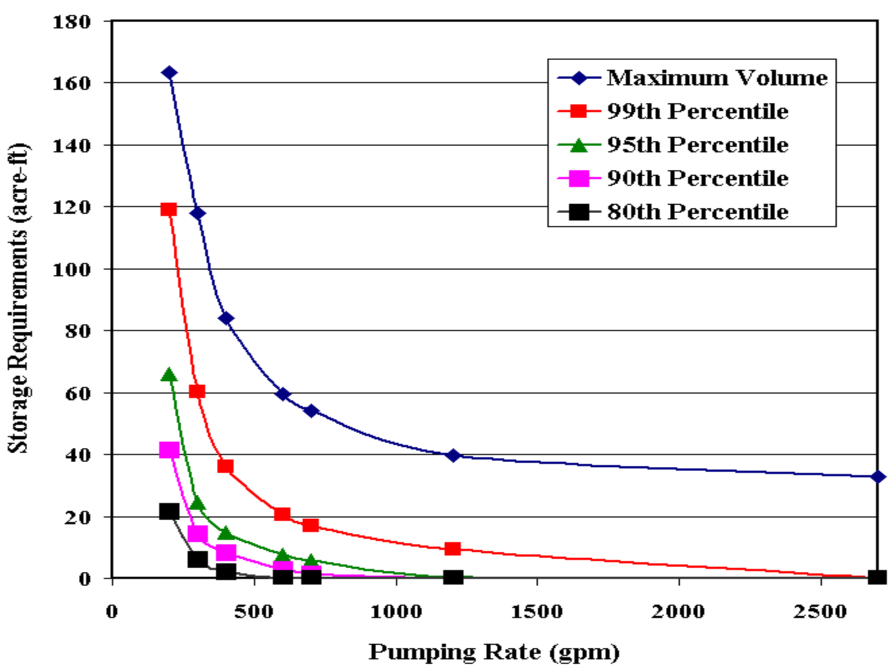

Figure 11. Storage volume requirements versus WWTP standby capacity, 2-acre equalization basin

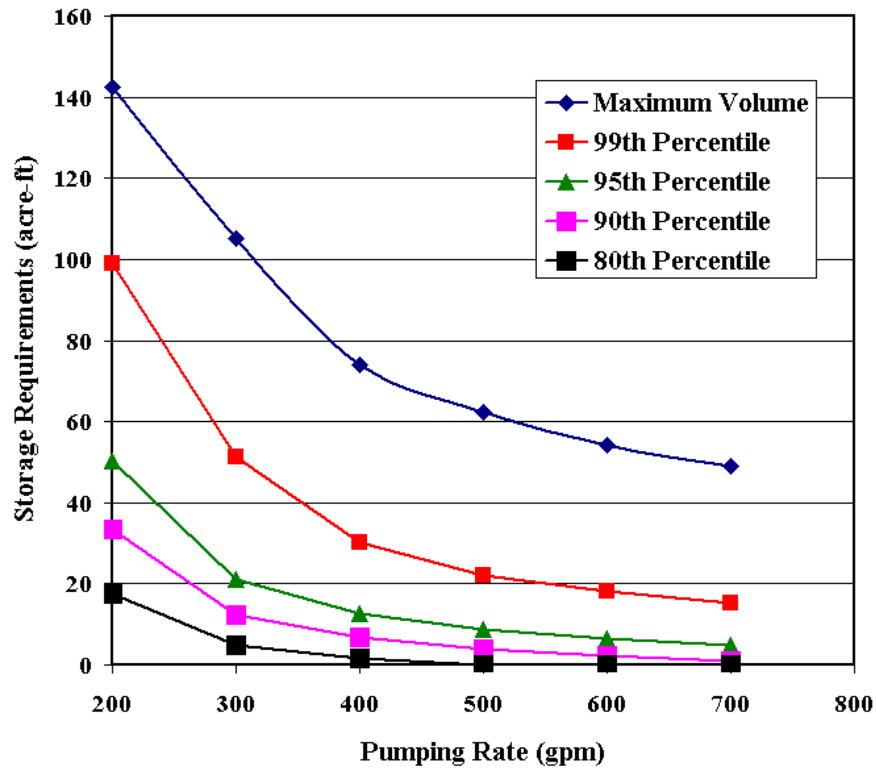

Figure 12. Storage volume requirements versus WWTP standby capacity, 10-acre equalization basin

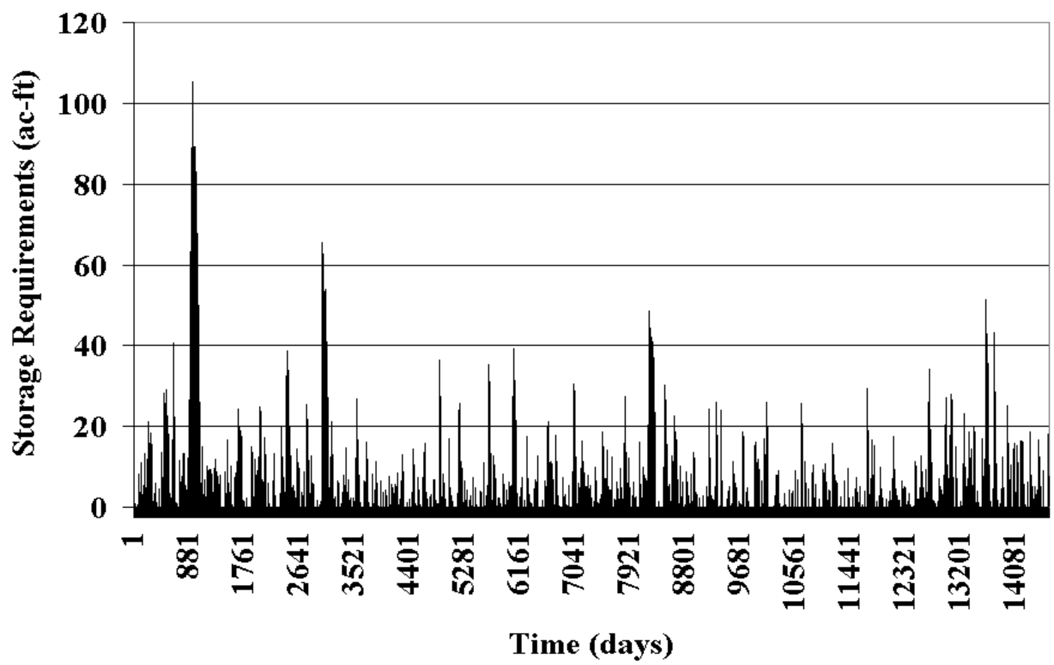

Figure 13. Runoff storage volume analysis for 10-acre equalization basin and a constant discharge rate of 300 gpm (hydraulic dredging alternatives)

Consolidation flows for all alternatives were based on volume changes indicated by consolidation analysis. Table 6 summarizes the average and peak combined flows (groundwater, effluent, runoff and consolidation) for the three alternatives evaluated. Tables 7 and 8 summarize the character of the mean and peak individual and combined flows, based on the flow assumptions contained in Table 6. Estimated constituent concentrations in the contributing aqueous streams were based on available historical data obtained from effluent testing, groundwater sampling, and partitioning theory. For some organic compounds (for example, ethylbenzene), predicted concentrations are much higher from one data source than from another, approaching or exceeding solubility limits. For groundwater samples, evidence suggests that in some cases free product contributed to high sample concentrations. The influence of high groundwater concentrations on the concentration of the combined flows is evident when comparing predicted concentrations for mechanical and hydraulic alternatives (Tables 7 and 8). Little dilution of groundwater occurs for the low flow mechanical dredging alternative, while substantial dilution occurs for the high flow hydraulic alternatives, resulting in higher concentrations for some constituents for the mechanical dredging alternative. During non-dredging periods, however, similar concentrations are predicted among the alternatives as a result of precipitation occurring over an equivalent area, diluting the groundwater with comparable volumes of surface runoff. Although elutriate testing was conducted in 1985, unexpectedly low values were obtained. Worst-case predictions were therefore used to estimate concentrations in combined waste streams; actual values may be considerably lower than those presented here. For the purpose of a comparative analysis, these worst-case predictions are appropriate. However, at the same time, these predictions are not meant to be used to represent actual conditions. Flow rate and concentration calculations are further described in Appendix B.

The period of time available for drawdown will largely determine whether drawdown requirements will govern treatment plant capacity. If sufficient time is available, differences in soil storage capacity and drawdown depth can be accommodated with lower treatment plant capacity. For the mechanical dredging alternative, plant capacity is governed by storm flows, unless equalization is provided. A minimum WWTP capacity of 200 gpm is necessary to accommodate a reasonable drawdown capability and keep equalization basin requirements within reasonable values. Maximum treatment plant capacity is required for the hydraulic dredging scenarios, with the peak flow estimated at roughly 2700 gpm.

Transfer of water from the primary cells to the equalization basin and from the equalization basin to the WWTP will be accomplished by pumping. Table 3 (Annualized Estimated Pumping Requirements) summarizes volumes and pump capacity requirements.

Table 6. Wastewater Treatment Plant Flows

Flow Component	Mean Flows			Alternative Condition	Combined Mean Flows					
	Alt 1	Alt 2	Alt 3		Alt 1		Alt 2		Alt 3	
	cfs	cfs	cfs		cfs	gpm	cfs	gpm	cfs	gpm
1. Qeff hydraulic net [1, 2]	N/A	4.0	4.2	A: Hydraulic Dredging - Streams 1 + 3 + 4 + 6	N/A	N/A	4.2	1900	4.4	2000
2. Qeff mechanical	0.000	N/A	N/A	B: Mechanical Dredging - Streams 2 + 3 + 4 + 6	0.071	25	N/A	N/A	N/A	N/A
3. QGW	0.002	0.002	0.002	C: Off Season Period - Streams 3 + 5 + 6	0.32	140	0.41	180	0.41	180
4. Qnet precip 4/15-10/15 mean	0.014	0.014	0.014							
5. Qnet precip 10/16-4/14 mean	0.26	0.26	0.26							
6. Qnet consol mean [3]	0.055	0.15	0.15							

Flow Component	Peak Flows			Alternative Condition	Combined Peak Flows					
	Alt 1	Alt 2	Alt 3		Alt 1		Alt 2		Alt 3	
	cfs	cfs	cfs		cfs	gpm	cfs	gpm	cfs	gpm
7. Qeff hydraulic actual [1, 4]	N/A	5.9	5.9	D: Hydraulic Dredging - Streams 7 + 9 + 10 + 12	N/A	N/A	6.2	2800	6.1	2700
8. Qmechanical	0.000	N/A	N/A	E: Mechanical Dredging - Streams 8 + 9 + 10 +12	0.12	53	N/A	N/A	N/A	N/A
9. Q GW	0.002	0.002	0.002	F: Off Season Period - Streams 9 + 11 + 12	0.36	160	0.52	240	0.47	210
10. Qnet precip 4/15-10/15 mean	0.014	0.014	0.014							
11. Qnet precip 10/16-4/14 mean	0.26	0.26	0.26							
12. Qnet consol peak annual [5]	0.10	0.26	0.21							

[1] Assumes production of a 12-inch hydraulic dredge without boosters; flow rate would be less under average conditions with booster pumps.
[2] Qeff hydraulic net is the average rate of effluent generation during a dredging project computed as the total slurry volume pumped into the CDF during the dredging period less the new volume of settle solids retained in the primary settling basin during the dredging period divided by the dredging period.
[3] Qnet consol mean is the average annual consolidation flow for the period of consideration (26 years for Alternative 3 and 32 years f or Alternatives 1 and 2, respectively). This consolidation flow rate is not necessarily continuous for 365 days per year, but rather reflects the peak additive effects of consolidation from previously placed layers and the current layer in a single year.
[4] Qeff hydraulic actual is the average dredge discharge rate during the dredging period.
[5] Qnet consol peak annual is the maximum annual consolidation flow for the period of consideration (26 years for Alternative 3 and 32 years f or Alternatives 1 and 2, respectively).

Table 7. Mean Flow Concentrations						
Analyte	**Mean Flow Concentration during Dredging (ug/l)**			**Mean Flow Concentration in Off Season (ug/l)**		
	Alt 1	**Alt 2**	**Alt 3**	**Alt 1**	**Alt 2**	**Alt 3**
	Cond. B [1]	**Cond. A [2]**	**Cond. A [2]**	**Cond. C [3]**	**Cond. C [3]**	**Cond. C [3]**
Metals						
Aluminum	40	0.63	0.50	7.5	6.0	6.4
Arsenic	75	8.0	6.0	14	41	27
Barium	3.3	0.052	0.041	0.62	0.50	0.53
Boron	11	0.18	0.14	2.1	1.7	1.8
Cadmium	20	3.3	2.8	5.5	12	8.7
Chromium	9.3	0.15	0.12	1.7	1.4	1.5
Chromium (hex)	96	39	37	64	76	69
Copper	32	34	35	25	25	25
Iron	3700	3900	4000	1800	2700	2200
Lead	300	70	68	110	130	120
Magnesium	1900	29	23	350	280	300
Manganese	10	0.16	0.12	1.9	1.5	1.6
Mercury	0.15	0.0039	0.0025	0.028	0.038	0.032
Nickel	62	32	32	36	36	35
Thallium	1.4	0.022	0.018	0.27	0.22	0.23
Zinc	600	440	430	400	460	430
PAH's						
2-Methylnaphthalene	1400	22	17	260	210	220
Acenaphthene	180	120	120	43	69	55
Acenaphthylene	61	47	49	13	25	19
Anthracene	46	25	26	11	16	13
Benzo(a)Anthracene	2.2	3.0	3.1	0.41	1.2	0.81
Benzo(a)Pyrene	0.80	1.1	1.1	0.15	0.44	0.29
Benzo(b)Fluoranthene	1.2	1.6	1.6	0.21	0.64	0.42
Benzo(G,H,I)Perylene	2.9	4.0	4.1	0.55	1.6	1.1
Benzo(k)Fluoranthene	1.2	1.6	1.6	0.21	0.64	0.42
Chrysene	1.9	2.6	2.7	0.36	1.1	0.71
Dibenzofuran	57	0.90	0.71	11	8.6	9.1
Fluoranthene	18	24	25	3.3	9.8	6.4
Fluorene	45	57	58	16	29	23
Indeno(1,2,3-C,D)Pyrene	0.15	0.21	0.21	0.028	0.084	0.056
Naphthalene	9700	13000	13000	1900	5400	3600
Phenanthrene	330	83	84	69	79	71
Pyrene	11	7.6	7.8	2.0	3.8	2.8

[1] Mechanical Dredging - Streams 2 + 3 + 4 + 6 (Table 6 Condition B)
[2] Hydraulic Dredging - Streams 1 + 3 + 4 + 6 (Table 6 Condition A)
[3] Off Season Period - Streams 3 + 5 + 6 (Table 6 Condition C)

(Continued)

CONSTITUENT CONCENTRATIONS IN AQUEOUS STREAMS WERE ESTIMATED FOR THE PURPOSES OF ALTERNATIVES COMPARISON AND PRELIMINARY EVALUATION OF TREATMENT REQUIREMENTS USING MULTIPLE ANECDOTAL SOURCES AND PARTITIONING ANALYSIS. TREATABILITY STUDIES ARE PLANNED TO OBTAIN A CURRENT DATA SET REPRESENTATIVE OF ACTUAL EXPECTED CONDITIONS, WHICH WILL BE USED IN THE DESIGN OF THE WWTP.

Table 7. Mean Flow Concentrations

Analyte	Mean Flow Concentration during Dredging (ug/l)			Mean Flow Concentration in Off Season (ug/l)		
	Alt 1	Alt 2	Alt 3	Alt 1	Alt 2	Alt 3
	Cond. B [1]	Cond. A [2]	Cond. A [2]	Cond. C [3]	Cond. C [3]	Cond. C [3]
Chlorinated Pesticides						
p,p'DDE	0.0099	0.0013	0.0001	0.034	0.024	0.029
Aldrin	0.0061	0.0082	0.0085	0.0011	0.0034	0.0022
Alpha-BHC	0.018	0.0003	0.0002	0.0033	0.0027	0.0028
Chlordane, Technical	0.025	0.0004	0.0003	0.0047	0.0038	0.0040
Heptachlor	0.027	0.0004	0.0003	0.0050	0.0041	0.0043
Semivolatile Organics						
Phenol	840	510	510	160	460	310
Inorganic/General Chemistry						
Carbon, Total Organic	290000	58000	52000	57000	160000	110000
Cyanide	130	8.3	6.9	44	43	42
Ammonia-N	420000	110000	100000	85000	240000	160000
Nitrogen, Nitrate	51	0.79	0.63	9.4	7.7	8.1
Phosphorus, Total	3.6	0.06	0.04	0.67	0.54	0.57
Oil and Grease	30000	11000	10000	21000	20000	20000
Sulfate	29000	64000	65000	56000	41000	49000
TDS	270000	360000	365000	140000	190000	160000
PCB Aroclors						
PCB (Aroclor-1248)	0.78	0.60	0.57	1.4	1.1	1.2
PCB (Aroclor-1254)	0.13	0.0021	0.0016	0.025	0.0200	0.021
Volatile Organic Compounds						
Acetone	18	0.29	0.23	3.4	2.8	2.9
Benzene	2000	32	25	380	310	320
Chlorobenzene	0.72	0.011	0.0089	0.13	0.11	0.11
Chloroform	0.014	0.0002	0.0002	0.0027	0.0022	0.0023
Ethyl Benzene	21000	330	260	3900	3200	3300
Methylene chloride	6.4	0.10	0.080	1.2	0.97	1.0
1,1,2,2-Tetrachloroethane	5.7	0.09	0.071	1.1	0.87	0.91
Toluene	9300	150	120	1700	1400	1500
1,1,2-Trichloroethane	3.9	0.062	0.049	0.73	0.59	0.62
Xylenes (total)	31000	490	390	5900	4800	5000

[1] Mechanical Dredging - Streams 2 + 3 + 4 + 6 (Table 6 Condition B)

[2] Hydraulic Dredging - Streams 1 + 3 + 4 + 6 (Table 6 Condition A)

[3] Off Season Period - Streams 3 + 5 + 6 (Table 6 Condition C) **(Concluded)**

CONSTITUENT CONCENTRATIONS IN AQUEOUS STREAMS WERE ESTIMATED FOR THE PURPOSES OF ALTERNATIVES COMPARISON AND PRELIMINARY EVALUATION OF TREATMENT REQUIREMENTS USING MULTIPLE ANECDOTAL SOURCES AND PARTITIONING ANALYSIS. TREATABILITY STUDIES ARE PLANNED TO OBTAIN A CURRENT DATA SET REPRESENTATIVE OF ACTUAL EXPECTED CONDITIONS, WHICH WILL BE USED IN THE DESIGN OF THE WWTP.

Table 8. Peak Flow Concentrations						
Analyte	**Peak Flow Concentration during Dredging (ug/l)**			**Peak Flow Concentration in Off Season (ug/l)**		
	Alt 1	**Alt 2**	**Alt 3**	**Alt 1**	**Alt 2**	**Alt 3**
	Cond. E [1]	**Cond. D [2]**	**Cond. D [2]**	**Cond. F [3]**	**Cond. F [3]**	**Cond. F [3]**
Metals						
Aluminum	12	0.36	0.37	5.3	4.3	5.3
Arsenic	95	8.3	6.6	40	52	39
Barium	1.0	0.030	0.031	0.44	0.36	0.44
Boron	3.6	0.10	0.11	1.5	1.2	1.5
Cadmium	25	3.4	3.0	12	15	12
Chromium	2.9	0.084	0.086	1.2	0.99	1.2
Chromium (hex)	100	38	37	75	80	75
Copper	25	34	35	24	24	24
Iron	4300	4000	4000	2600	3000	2600
Lead	200	69	68	120	130	120
Magnesium	580	17	17	250	200	250
Manganese	3.1	0.091	0.092	1.3	1.1	1.3
Mercury	0.083	0.0033	0.0024	0.035	0.038	0.035
Nickel	43	32	32	35	35	35
Thallium	0.45	0.013	0.013	0.19	0.15	0.19
Zinc	580	440	430	450	480	450
PAH's						
2-Methylnaphthalene	440	13	13	190	150	190
Acenaphthene	140	120	120	67	77	66
Acenaphthylene	53	49	49	24	29	24
Anthracene	32	26	26	15	17	15
Benzo(a)Anthracene	2.8	3.1	3.1	1.2	1.6	1.2
Benzo(a)Pyrene	1.0	1.1	1.1	0.43	0.56	0.42
Benzo(b)Fluoranthene	1.5	1.6	1.6	0.62	0.81	0.61
Benzo(G,H,I)Perylene	3.7	4.1	4.1	1.6	2.1	1.6
Benzo(k)Fluoranthene	1.5	1.6	1.6	0.62	0.81	0.61
Chrysene	2.4	2.7	2.7	1.0	1.4	1.0
Dibenzofuran	18	0.52	0.53	7.6	6.1	7.6
Fluoranthene	22	25	25	9.5	12.4	9.4
Fluorene	54	58	58	29	34	28
Indeno(1,2,3-C,D)Pyrene	0.19	0.21	0.21	0.082	0.17	0.081
Naphthalene	12000	13000	13000	5300	6800	5200
Phenanthrene	160	84	84	73	74	73
Pyrene	8.6	7.8	7.8	3.7	4.4	3.6

[1] Mechanical Dredging - Streams 8 + 9 + 10 +12 (Table 6 Condition E)
[2] Hydraulic Dredging - Streams 7 + 9 + 10 + 12 (Table 6 Condition D)
[3] Off Season Period - Streams 9 + 11 + 12 (Table 6 Condition F)

(Continued)

CONSTITUENT CONCENTRATIONS IN AQUEOUS STREAMS WERE ESTIMATED FOR THE PURPOSES OF ALTERNATIVES COMPARISON AND PRELIMINARY EVALUATION OF TREATMENT REQUIREMENTS USING MULTIPLE ANECDOTAL SOURCES AND PARTITIONING ANALYSIS. TREATABILITY STUDIES ARE PLANNED TO OBTAIN A CURRENT DATA SET REPRESENTATIVE OF ACTUAL EXPECTED CONDITIONS, WHICH WILL BE USED IN THE DESIGN OF THE WWTP.

Table 8. Peak Flow Concentrations

Analyte	Peak Flow Concentration during Dredging (ug/l)			Peak Flow Concentration in Off Season (ug/l)		
	Alt 1	Alt 2	Alt 3	Alt 1	Alt 2	Alt 3
	Cond. E [1]	Cond. D [2]	Cond. D [2]	Cond. F [3]	Cond. F [3]	Cond. F [3]
Chlorinated Pesticides						
p,p'DDE	0.0031	0.0001	0.0001	0.024	0.020	0.025
Aldrin	0.0078	0.0085	0.0085	0.0033	0.0043	0.0032
Alpha-BHC	0.0056	0.0002	0.0002	0.0024	0.0019	0.0024
Chlordane, Technical	0.0078	0.0002	0.0002	0.0033	0.0027	0.0033
Heptachlor	0.0084	0.0002	0.0002	0.0035	0.0029	0.0036
Semivolatile Organics						
Phenol	1100	520	510	450	590	440
Inorganic/General Chemistry						
Carbon, Total Organic	360000	60000	54000	160000	200000	150000
Cyanide	63	7.1	6.7	41	39	41
Ammonia-N	530000	110000	110000	230000	300000	230000
Nitrogen, Nitrate	16	0.46	0.47	6.7	5.4	6.8
Phosphorus, Total	1.1	0.03	0.03	0.47	0.38	0.47
Oil and Grease	21000	10000	10000	20000	19000	20000
Sulfate	15000	64000	65000	42000	35000	42000
TDS	290000	360000	360000	190000	210000	180000
PCB Aroclors						
PCB (Aroclor-1248)	0.64	0.57	0.57	1.1	1.0	1.1
PCB (Aroclor-1254)	0.041	0.0012	0.0012	0.018	0.014	0.018
Volatile Organic Compounds						
Acetone	5.7	0.17	0.17	2.4	2.0	2.4
Benzene	630	18	19	270	220	270
Chlorobenzene	0.22	0.0065	0.0066	0.094	0.076	0.095
Chloroform	0.0045	0.0001	0.0001	0.0019	0.0015	0.0019
Ethyl Benzene	6600	190	190	2800	2300	2800
Methylene chloride	2.0	0.058	0.059	0.85	0.69	0.86
1,1,2,2-Tetrachloroethane	1.8	0.052	0.053	0.76	0.61	0.76
Toluene	2900	84	86	1300	990	1200
1,1,2-Trichloroethane	1.2	0.036	0.036	0.52	0.42	0.52
Xylenes (total)	9800	290	290	4200	3400	4200

[1] Mechanical Dredging - Streams 8 + 9 + 10 +12 (Table 6 Condition E)

[2] Hydraulic Dredging - Streams 7 + 9 + 10 + 12 (Table 6 Condition D)

[3] Off Season Period - Streams 9 + 11 + 12 (Table 6 Condition F) **(Concluded)**

CONSTITUENT CONCENTRATIONS IN AQUEOUS STREAMS WERE ESTIMATED FOR THE PURPOSES OF ALTERNATIVES COMPARISON AND PRELIMINARY EVALUATION OF TREATMENT REQUIREMENTS USING MULTIPLE ANECDOTAL SOURCES AND PARTITIONING ANALYSIS. TREATABILITY STUDIES ARE PLANNED TO OBTAIN A CURRENT DATA SET REPRESENTATIVE OF ACTUAL EXPECTED CONDITIONS, WHICH WILL BE USED IN THE DESIGN OF THE WWTP.

Volatile Emissions

The relative flux rates for constituents of concern were calculated for representative conditions at the CDF for both accelerated hydraulic and mechanical dredging scenarios, assuming unit concentrations of the constituents of concern in the sediments and corresponding predicted concentrations in effluent, runoff and consolidation flows. The volatile emission rates for the standard hydraulic dredging scenario would fall between the accelerated hydraulic and mechanical dredging scenarios. Since the ponded conditions of dredging for the standard dredging scenario exist only one-third of the days as for the accelerated dredging scenario, the volatile emission rates of the standard dredging scenario would be expected to be somewhat more similar to the rates of the mechanical dredging scenario than the accelerated dredging scenario. The partitioning coefficients and mass transfer coefficients upon which the ponded conditions are based are considered to be conservative, reflecting worst-case water concentrations. Actual conditions may differ from those assumed for the purposes of the comparative analysis and are not intended to represent the actual magnitude of emissions expected.

Mass transfer rates differ between dry, wet and ponded surfaces. To estimate flux rates, assumptions must be made regarding surface conditions and areas. Equalization basins were assumed to be ponded at all times. The chemistry of the water in the equalization basin would be expected to be dominated by effluent during hydraulic disposal and by runoff and consolidation flows in the off season. Also, runoff from recently placed, unoxidized material would be different in character to runoff from dried, oxidized material. Concentrations in ponded areas and emissions from ponded areas and exposed dredged material surfaces were estimated based on representative conditions for hydraulic and mechanical disposal. (See Appendix C for more detailed information.) Figures 14 through 19 illustrate the assumed surface conditions for normal and peak operations and off-season periods. Figure 14 illustrates peak operating conditions for the mechanical dredging alternative, when two primary cells would be filled in one year (unoxidized conditions) and the other primary cell and TSCA cell would be idle (oxidized conditions). Figure 15 illustrates normal operating conditions for the mechanical dredging alternative, when one primary cell would be filled during the year and the remaining cells would be idle. Figure 16 illustrates off-season conditions for the mechanical dredging alternative, when no dredging is actively occurring, and all material is assumed to be in an oxidized condition.

Figure 17 illustrates peak operating conditions for the hydraulic dredging alternatives, when two primary cells and the TSCA cell are assumed to be ponded during active disposal operations, and the remaining primary cell is inactive and assumed to be oxidized. Figure 18 illustrates normal operating conditions for the hydraulic dredging alternatives, when one primary cell and the TSCA cell are assumed to be ponded during active disposal operations, a second primary cell is drained but unoxidized, and the third primary cell is assumed to be drying and oxidized. Figure 19 illustrates off-season conditions for the hydraulic dredging alternatives, in which the TSCA cell is assumed to

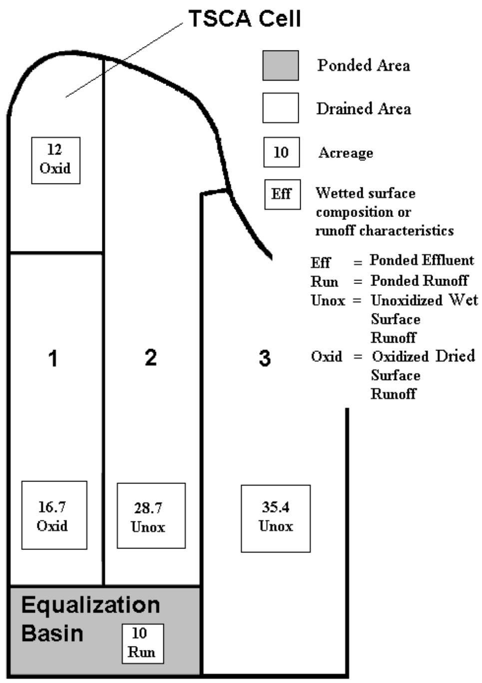

Figure 14. Alternative 1 (mechanical) surface conditions during peak operation

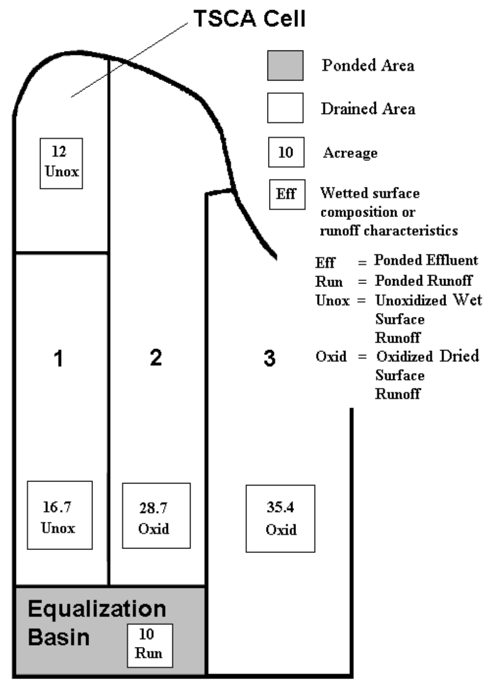

Figure 15. Alternative 1 (mechanical) surface conditions during normal operation

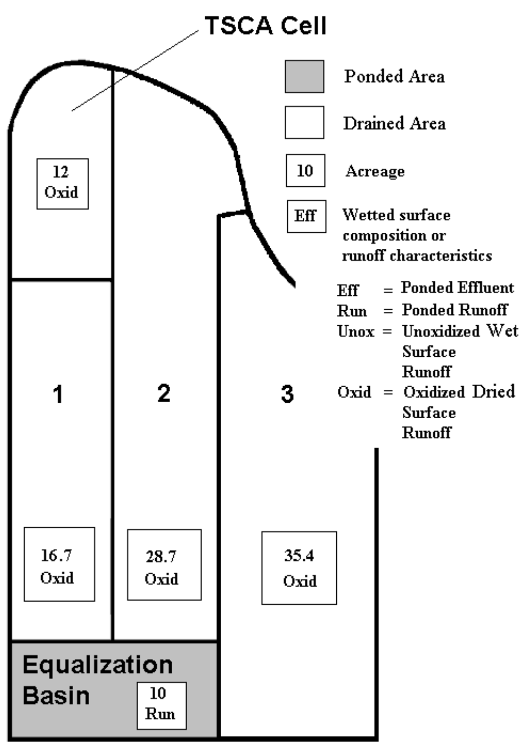

Figure 16. Alternative 1 (mechanical) surface conditions during off season

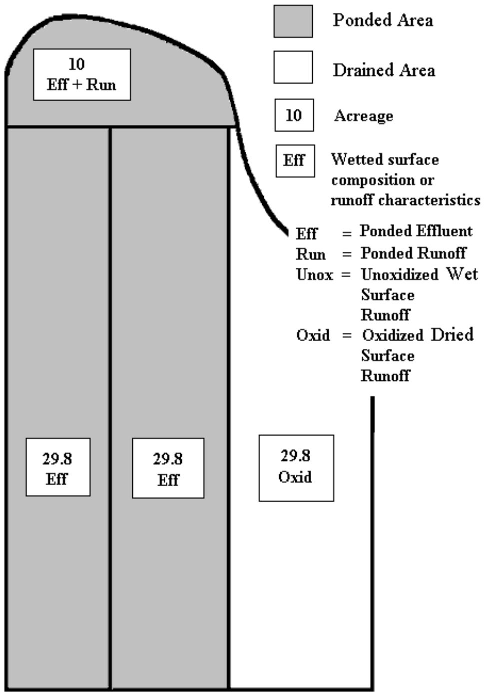

Figure 17. Alternative 3 (hydraulic) surface conditions during peak operation

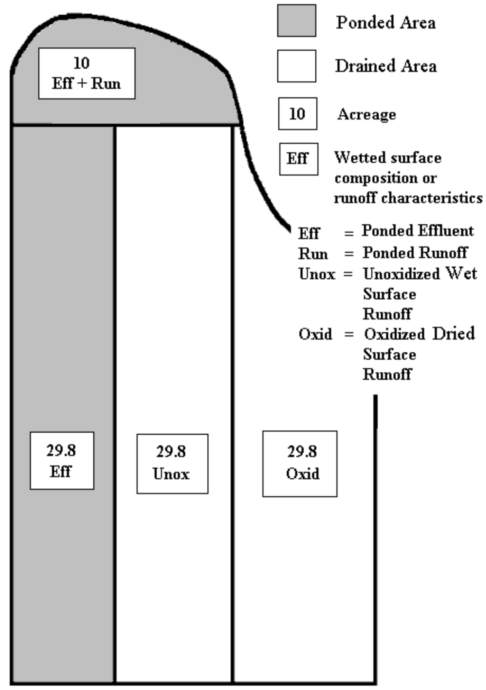

Figure 18. Alternative 3 (hydraulic) surface conditions during normal operation

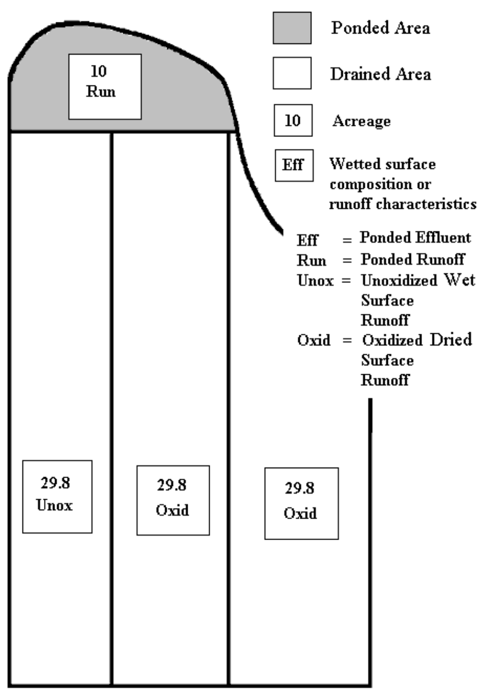

Figure 19. Alternative 3 (hydraulic) surface conditions during off season

be utilized for equalization and is ponded, the most recently used primary cell is drained but unoxidized, and the remaining two primary cells are dry and in an oxidized condition.

Table 9 summarizes the relative flux rates under each of these conditions for all constituents of concern, given a unit sediment contaminant concentration (1 mg/kg). The flux rates show differences in volatilization among contaminants. The annual volatile emission losses for the mechanical and accelerated hydraulic dredging alternatives are given and compared in Table 10. Generally, flux rates are comparable for mechanical and hydraulic disposal; however, the fluxes for constituents having low partitioning coefficients are higher for the hydraulic alternatives. Only 20% of the constituents differ by more than 10% of the median value; all of these constituents have higher volatile fluxes for hydraulic dredging alternatives than for mechanical dredging alternatives. Only five ratios of hydraulic to mechanical volatile losses are as high as 1.48 (1.48, 1.58, 1.92, 3.48 and 7.39).

Particulate Emissions

Particulate emissions (losses of fugitive dust) for the various alternatives will be a function of the area present in an exposed, unvegetated, dried condition. The areas in ponded, wet drained, and dry drained conditions for the accelerated hydraulic dredging and mechanical dredging alternatives are shown in Figures 14 through 19. For the mechanical dredging alternative, the surface of the three primary cells (92.8 acres) should be somewhat dried during the 6-month off season, but limited fugitive dust is expected during this period since typically the precipitation exceeds the evaporation during the off season. During this same period, the surface of the two primary cells (59.0 acres) for the accelerated hydraulic dredging alternative should be somewhat dried. As such, the tendency for fugitive dust production during the off season for the mechanical dredging alternative may be about 1.6 times as large as fugitive dust production for accelerated hydraulic dredging.

During the mechanical dredging season, the surface of one primary cell (28.7 acres) should be dried for 3 months and the surface of two primary cells (57.4 acres) should be dried for the other 3 months, a total of 258.3 acre-months of exposure. During the accelerated hydraulic dredging season, the surface of one primary cell (29.8 acres) should be dried for 6 months, a total of 179 acre-months of exposure. As such, the tendency for fugitive dust production for the mechanical dredging alternative during the 6-month dredging season may be about 1.5 times as large as fugitive dust production for accelerated hydraulic dredging.

In addition to losses from the exposed surfaces, losses can also occur from the dredged material transfer operations. Mechanical transfer operations and trucking are subject to much greater losses than hydraulic pipeline operations. The losses are specific to the design of the transfer systems. With good management, such as truck washing and appropriate loss controls, particulate emissions from the transfer operations can be

Table 9. Volatile Fluxes and Emissions for Mechanical and Accelerated Hydraulic Dredging Alternatives for Unit Sediment Contaminant Concentration

Analyte	Ponded			Exposed
	Accelerated Hydraulic		Mechanical	Both
	Flux Normal & Peak (kg/ac/day)	Flux Off Season (kg/ac/day)	Flux Normal, Peak & Off Season (kg/ac/day)	Flux During Dredging & Off Season (kg/ac/day)
Mercury	0.00019	0.0000093	0.000065	0.0011
2-Methylnaphthalene	0.00139	0.00033	0.0010	0.00012
Acenaphthene	0.00119	0.00037	0.0011	0.00018
Acenaphthylene	0.0022	0.00032	0.00079	0.00013
Anthracene	0.00022	0.00012	0.00020	0.000028
Benzo(a)Anthracene	0.0000016	0.0000015	0.0000016	0.0000012
Benzo(a)Pyrene	0.00000069	0.00000068	0.00000069	0.00000060
Benzo(b)Fluoranthene	0.0000059	0.0000058	0.0000059	0.0000046
Benzo(G,H,I)Perylene	0.0000067	0.0000056	0.0000066	0.0000033
Benzo(k)Fluoranthene	0.0000059	0.0000058	0.0000059	0.0000046
Chrysene	0.0000013	0.0000012	0.0000013	0.00000082
Dibenzo(a,h)anthracene	0.0000000016	0.0000000016	0.0000000016	0.0000000020
Dibenzofuran	0.00038	0.00014	0.00033	0.000031
Fluoranthene	0.000051	0.000040	0.000050	0.000011
Fluorene	0.00067	0.00028	0.00059	0.00010
Indeno(1,2,3-C,D)Pyrene	0.000000012	0.000000012	0.000000012	0.000000014
Naphthalene	0.0077	0.00056	0.0034	0.0010
Phenanthrene	0.00023	0.00013	0.00021	0.000031
Pyrene	0.000012	0.000011	0.000012	0.0000041
Benzene	0.090	0.00079	0.0075	0.041
Ethylbenzene	0.010	0.00064	0.0042	0.021
Tetrachloroethylene	0.015	0.00053	0.0041	0.054
Trichloroethylene	0.042	0.00061	0.0055	0.080
Toluene	0.030	0.00071	0.0060	0.052
Xylene	0.011	0.00064	0.0043	0.031
4,4'-DDD	0.0000044	0.0000044	0.0000044	0.0000029
o,p'-DDE	0.0000098	0.0000093	0.0000097	0.0000042
p,p'DDT	0.0000052	0.0000051	0.0000052	0.0000039
Aldrin	0.0000025	0.0000025	0.0000025	0.0000084
Chlordane, Technical	0.000095	0.000069	0.000092	0.0000061
Dieldrin	0.0019	0.00024	0.0011	0.000029
Endrin	0.000011	0.0000069	0.000010	0.0000010
Gamma-BHC (Lindane)	0.000047	0.000034	0.000045	0.0000024
Heptachlor Epoxide	0.013	0.00034	0.0028	0.00020
Phenol	0.012	0.000050	0.00049	0.00038
Dioxin	0.0000013	0.0000013	0.0000013	0.0000015
Furan	0.0000035	0.0000035	0.0000035	0.0000033
PCB (Aroclor-1248)	0.000016	0.000015	0.000016	0.000072
PCB (Aroclor-1254)	0.0000078	0.0000077	0.0000078	0.000038
PCB Total	0.000016	0.000015	0.000016	0.000066

THE VOLATILE EMISSIONS VALUES GIVEN IN THIS TABLE ARE NOT INTENDED TO REPRESENT A COMPREHENSIVE EVALUATION OF EMISSIONS FROM THE DISPOSAL SITE; RATHER THEY ARE INTENDED STRICTLY FOR THE PURPOSES OF ALTERNATIVES COMPARISON, ASSUMING WORST-CASE OPERATING CONDITIONS AND UNIT CONTAMINANT CONCENTRATIONS IN THE SEDIMENT (1 MG/KG). ACTUAL EMISSIONS WILL BE EVALUATED AND ADDRESSED IN A MORE COMPREHENSIVE RISK ASSESSMENT.

Table 10. Comparison of Annual Volatile Emission Losses

Analyte	Annual Volatile Losses for Accelerated Hydraulic Dredging Alternative (kg)	Annual Volatile Losses for Mechanical Dredging Alternative (kg)	Ratio of Annual Volatile Losses for Accelerated Hydraulic Alternative to Mechanical Dredging Alternative
Mercury	30.7	29.9	1.03
2-Methylnaphthalene	14.6	13.4	1.09
Acenaphthene	14.5	15.4	0.94
Acenaphthylene	21.7	11.3	1.92
Anthracene	2.71	2.73	0.99
Benzo(a)Anthracene	0.0463	0.0464	1.00
Benzo(a)Pyrene	0.0225	0.0225	1.00
Benzo(b)Fluoranthene	0.178	0.178	1.00
Benzo(G,H,I)Perylene	0.152	0.153	1.00
Benzo(k)Fluoranthene	0.178	0.178	1.00
Chrysene	0.0342	0.0342	1.00
Dibenzo(a,h)anthracene	0.000068	0.000068	1.00
Dibenzofuran	4.16	4.06	1.02
Fluoranthene	0.778	0.785	0.99
Fluorene	8.62	8.53	1.01
Indeno(1,2,3-C,D)Pyrene	0.000467	0.000467	1.00
Naphthalene	89.7	60.4	1.48
Phenanthrene	2.93	2.95	0.99
Pyrene	0.221	0.222	1.00
Benzene	1810.	1150.	1.58
Ethylbenzene	640.	596.	1.07
Tetrachloroethylene	1540.	1450.	1.06
Trichloroethylene	2420.	2130.	1.14
Toluene	1600.	1420.	1.13
Xylene	891.	843.	1.06
4,4'-DDD	0.119	0.119	1.00
o,p'-DDE	0.207	0.207	1.00
p,p'DDT	0.152	0.152	1.00
Aldrin	0.243	0.243	1.00
Chlordane, Technical	1.06	1.07	0.99
Dieldrin	16.8	12.2	1.38
Endrin	0.127	0.128	0.99
Gamma-BHC (Lindane)	0.507	0.514	0.99
Heptachlor Epoxide	115.	33.0	3.48
Phenol	110.	14.9	7.39
Dioxin	0.051	0.051	1.00
Furan	0.119	0.119	1.00
PCB (Aroclor-1248)	2.04	2.04	1.00
PCB (Aroclor-1254)	1.06	1.06	1.00
PCB Total	1.86	1.86	1.00
		Mean	1.29
		Minimum	0.94
		Median	1.00
		Maximum	7.39

THE VOLATILE EMISSIONS VALUES GIVEN IN THIS TABLE ARE NOT INTENDED TO REPRESENT A COMPREHENSIVE EVALUATION OF EMISSIONS FROM THE DISPOSAL SITE; RATHER THEY ARE INTENDED STRICTLY FOR THE PURPOSES OF ALTERNATIVES COMPARISON, ASSUMING WORST-CASE OPERATING CONDITIONS AND UNIT CONTAMINANT CONCENTRATIONS IN THE SEDIMENT (1 MG/KG). ACTUAL EMISSIONS WILL BE EVALUATED AND ADDRESSED IN A MORE COMPREHENSIVE RISK ASSESSMENT.

minimized, and should be much less than losses from the CDF, where the area is much larger and controls more difficult to implement.

Particulate emissions can be minimized with the use of controls such as physical barriers, chemical stabilizers, or vegetation (Francingues et al. 1985, US Army Corps of Engineers 1983). Physical barriers may include fibers, mulches or geotextiles. Issues of concern would be logistical feasibility of placement over unconsolidated material and cost. Chemical suppressants are also commercially available, but suitability for this application has not been determined. Ponding, or water spray systems may be utilized, but would prevent or delay the desired dewatering and consolidation of the material. Surface vegetation and windscreens can be a low-tech alternative to reducing particulate transport. Dewatering may be somewhat facilitated by plant transpiration, although this may be offset by surface shading. Since vegetation typically volunteers in idle cells of CDFs where salinity of the material is not high, it is expected that an effective surface vegetative cover could be established for particulate control. The unconsolidated condition of the dredged material and initially high water content may prevent immediate seeding of the material for both hydraulic and mechanical alternatives. Floating equipment could be used for seeding operations if necessary, as soon as the surface water content is conducive to germination. High concentrations of PAHs and zinc in the sediment may also inhibit seed germination without chemical additives.

3 - Cost Estimates

Planning-level cost estimates were prepared to provide an economic basis for comparison of the dredging and disposal alternatives. These cost estimates do not represent complete project construction cost estimates. Costs common to the three alternatives, such as groundwater containment measures, railroad relocation, air, groundwater and surface water sampling and testing, and dike maintenance, among others, were not included. Capital and/or operating costs were estimated for the following items for which significant differences were anticipated between the alternatives under consideration:

- Underwater survey
- Debris removal
- Dike construction
- Dredging
- Surface water pumping
- Suspended solids removal (flocculation) in the CDF
- Wastewater treatment
- Dredged material management

Cost estimates for some items were developed from lump sum and unit prices published in Appendix H of the DDR, which included indirect costs. All DDR cost estimates were adjusted to a January 2003 basis. For the cost estimates developed in 2003, indirect costs have been added to direct costs; 12% for overhead, 10% for profit, and 1% for bond. Next, contingency was applied to every item, ranging from 15 to 50%, with the majority falling within 15 to 25%. The level of design, cost of the individual component, and level of inherent risk associated with each item determine the contingency applied. This is more fully discussed in Appendix D. Costs were then converted to present value, taking into account the year in which expenditures will occur, and the applicable federal discount rate (PL 93-251 Section 80).

Major Cost Items

Underwater Survey/Debris Removal

As is the case with rivers in highly industrialized areas, it is expected that the Indiana Harbor Canal will contain large amounts of debris, such as iron pellets, shopping carts, cars, small appliances and other items. Debris removal can be efficiently accomplished during mechanical dredging, but can cause downtime and reduce efficiency of a hydraulic dredging operation. A debris survey and debris removal are therefore considered necessary prior to dredging for the hydraulic dredging alternatives. Side scan sonar and sub bottom profiler can be used to locate debris. A salvage contractor can then remove the debris using appropriate construction and salvage equipment (Randall 2000). Cost estimates were included for these two items.

Dike Construction

Dike construction is common to all three alternatives, but differences in storage requirements dictate different dike heights for the three alternatives, resulting in cost differentials between mechanical and hydraulic alternatives for a number of cost components. In the detailed cost breakdown, dike construction was separated into initial dike construction, dike raising, clay liner construction, and cap construction. Initial dike construction was assumed to be phased over 2 years, during which time all dikes will be constructed to a specified initial height. Dikes will be raised to their final height after backlog dredging is completed, before maintenance dredging begins. Although a clay liner is required on the interior face of the exterior dikes for all alternatives, the clay liner construction included in the cost estimate reflects only the additional cost associated with the construction of the liner for the hydraulic dredging alternatives, where dikes are higher and wider than for the mechanical dredging alternative. The DDR (USACE Chicago 2000) specifies dike construction using a combination of off-site materials and materials stripped from the site. As a simplifying assumption, for the cost estimate, all clay dike construction was assumed. The validity of this assumption will be verified in the detailed design phase and may impact the need for an additional clay liner. Capping of all cells was assumed to occur approximately 2 years after the final year of dredging.

Dredging

Cost to dredge the total project volume, including the cost to mobilize/demobilize each year of dredging, was based on a maximum hydraulic dredging rate of 334 cy/hr. This rate would be reduced to 270 cy/hr when 2 booster pumps are required, as when dredging in the channel sections farthest from the proposed disposal area. Dredging costs were higher for the mechanical dredging alternative than for the hydraulic dredging alternatives. Dredging cost for the standard hydraulic dredging is also higher than for the accelerated hydraulic dredging, reflecting additional mob/demob costs required to perform the dredging over a 25-year period, as opposed to a 19-year period.

Surface Water Pumping

Pumping will be required to maintain the groundwater gradient, transfer effluent and precipitation from primary disposal cells to the equalization basin, and transfer water from the equalization basin to the WWTP. Groundwater pumping requirements are assumed to be the same for all alternatives and are not included in this comparative analysis. Pumping rates from the disposal cells will occur at different rates, depending upon the time of year and dredging activity. Pumping requirements will be highest during hydraulic disposal and lowest during non-dredging periods. Table D2 in Appendix D summarizes the estimated pump capacity required to handle peak flows, average annual operating periods for these pumps, and capacity and operating periods for pumps required to handle average flows for non-dredging periods. Annual pumping volumes are the same as previously given in Table 3, but rates have been converted to equivalent, annualized values to provide a uniform basis for cost estimating.

Separate sets of pumps were assumed to handle effluent produced during dredging and storm flows during non-dredging periods. During hydraulic dredging, temporary transfer pumps will be utilized to transfer effluent to the equalization basin and from the equalization basin to the WWTP. These pumps will be part of the contract of the dredging company and will be removed after dredging is completed each year. No effluent transfer pumps are required for the mechanical dredging alternative. Standpipe/weir pumps will be permanently installed and will operate during lower flow conditions of the non-dredging season, transferring storm and consolidation flows from the disposal cells to the equalization basin and from the equalization basin to the WWTP. All alternatives will require standpipe/weir pumps; pump capacities will vary for the alternatives due to differences in consolidation flows, equalization basin size and WWTP capacity.

Wastewater Treatment

The cost estimate for the wastewater treatment plant was developed by Montgomery Watson Harza (MWH), a national leader in designing and constructing wastewater treatment plants, utilizing the design flows and contaminant concentrations presented in this report (WWTP flows and Appendix D). Plant capacity was based on peak flows: 2700 gpm for both hydraulic dredging alternatives, and 200 gpm for the mechanical dredging alternative. O&M costs were based on average seasonal flows (Appendix D). Capital costs included pumps, accessory tanks, and chemical feed systems for a complete system. The WWTP cost estimates are more fully described in Appendix D.

Operation and Maintenance Costs

The operation and maintenance activities associated with pre-closure (through capping of the CDF) were presented in Appendix H of the DDR. As for the capital costs, O&M costs were only developed for activities considered to vary significantly between alternatives. O&M costs were developed for water treatment, dredged material management, and standpipe/weir pumps, which replace the CDF surface water collection pumping presented in the DDR.

WWTP

Annual O&M costs for the wastewater treatment plant include:

- Labor
- Process chemicals
- Energy
- Building and equipment maintenance

Chemical costs were developed assuming typical dosages to estimate necessary quantities. Labor costs were assumed to vary seasonally, with less labor required during non-dredging periods in most cases. Additional discussion pertaining to the O&M cost basis for the WWTP is contained in Appendix D.

Dredged Material Management

Because the dredged material lift depths are relatively thin, adequate dewatering is assumed to occur without active dredged material management, with the exception of the first three years of operation for alternative 3. During accelerated hydraulic backlog dredging, lift depths are expected to range from approximately 7-10 feet. Surface trenching would be beneficial in facilitating dewatering of these lifts. Annual O&M costs were estimated for trenching for the first 3 years of alternative 3.

Pumping

The annual cost of operating, inspecting, maintaining and replacing the standpipe/weir pumps was included as an O&M cost. No O&M costs will be incurred for the transfer pumps, which will be part of the dredging contract and removed annually after dredging is completed each year.

Present Worth Analysis

Estimated costs for individual project components were converted to present value for comparison of the alternatives. The discount rate is established by the Office of Management and Budget for federal projects and is presently at 5 7/8%. The present value formula is:

$$PV = \frac{V_1}{(1+i)^1} + \frac{V_2}{(1+i)^2} + \frac{V_3}{(1+i)^3} + \ldots.. + \frac{V_n}{(1+i)^n}$$

where

i	= federal discount rate, %
n	= the period of consideration, years
V_i	= costs incurred in year i, constant 2003 $

Table 11 summarizes the results of the present worth analysis. Alternative 1, Mechanical Dredging, was the least cost alternative with a present value of $82,267,000, with contingency. Alternative 2 is the next least cost alternative with a present value of $98,755,000, and Alternative 3 is the highest cost alternative with a present value of $109,450,000.

Table 11. Alternatives Present Value Comparison with Contingency [1]			
Parameter	**Alternative 1 Mechanical (000s of dollars)**	**Alternative 2 Hydraulic (000s of dollars)**	**Alternative 3 Accel Hydraulic (000s of dollars)**
Construction and Dredging Activities			
Underwater survey	N/A	$73	$73
Debris removal	N/A	$3,449	$3,449
Year 1, cell construction	$5,902	$8,485	$14,573
Year 1, additional clay liner	N/A	$79	$109
Year 2, cell construction	$4,019	$6,725	$4,758
Year 2, additional clay liner	N/A	$63	$68
Standpipes/weirs	$220	$220	$220
Pumps in standpipes	$56	$80	$80
Raise dike heights	$6,637	$5,329	$4,906
Wastewater treatment plant (WWTP)	$2,877	$11,392	$11,392
Dredging	$41,890	$23,661	$28,732
Pumps during dredging	N/A	$3,501	$4,410
Cap	$2,594	$2,594	$3,654
Total Construction and Dredging Activities	**$64,196**	**$65,649**	**$76,423**
Operations and Maintenance Activities			
O&M of WWTP	$17,939	$32,846	$32,032
O&M for pumps in standpipes	$132	$260	$241
O&M for dredged material management	N/A	N/A	$754
Total Operations and Maintenance Activities	**$18,071**	**$33,106**	**$33,027**
COMPARISON TOTAL (000s)	**$82,267**	**$98,755**	**$109,450**

[1] A contingency, ranging from 15 to 50% with the majority within the 15 to 25% range, was applied individually at the line item level. The contingency assignment was based on the level of design detail, inherent risk associated with each item and the anticipated cost growth due to factors not yet identified at this time.

PLANNING-LEVEL COST ESTIMATES WERE PREPARED TO PROVIDE AN ECONOMIC BASIS FOR COMPARISON OF THE DREDGING AND DISPOSAL ALTERNATIVES. THESE COST ESTIMATES DO NOT REPRESENT COMPLETE PROJECT CONSTRUCTION COST ESTIMATES.

4 - Summary and Major Findings

The primary objective of the present study was to perform a planning-level evaluation of a limited number of dredging and placement alternatives for the operation of the Indiana Harbor and Canal Confined Disposal Facility (CDF), in order to address the public's desire, through comparative analysis, to re-evaluate the use of hydraulic dredging and disposal for this project. Mechanical dredging had been selected in a previous study, and a corresponding design for the CDF had been developed and documented in the DDR (USACE, Chicago 2000). The existing design documents specified completion of backlog dredging, which has been deferred since 1972, in 10 years. However, because hydraulic dredging potentially offered a means of expediting the backlog dredging, a planning-level comparison of three dredging alternatives was conducted: Alternative 1, mechanical dredging; Alternative 2, hydraulic dredging at the same rate as mechanical dredging; and Alternative 3, accelerated hydraulic dredging. Relative life cycle costs and air emissions for the different dredging alternatives and compatibility of hydraulic dredging with the already established design for the disposal site were key concerns.

Dredging and Disposal Operations

Evaluation of settling and storage requirements for hydraulic dredging indicates that hydraulic dredging is feasible with modifications to the CDF design specified in the DDR (USACE, Chicago 2000). A 12-inch hydraulic dredge appears to be the smallest hydraulic dredge that could accommodate the stated project depths. A 12-inch hydraulic dredge would permit completion of backlog dredging in 4 years, producing 14 hours/day, 6 days/week. While a larger dredge might permit a higher production rate, disposal site area constraints will ultimately limit production, and cost of suspended solids removal and WWTP capital cost can be reduced if a smaller dredge is used. Lift depths of the placed material will be lower for a lower production rate, facilitating dewatering. Modifications to the existing CDF design would include adjustment of dike heights and provision of 4 cells, with one serving as a flow equalization basin. Maximum required dike heights were estimated as follows: Alternative 1, mechanical dredging, 28.5 ft; Alternative 2, hydraulic dredging, 32.5 ft; and Alternative 3, accelerated hydraulic dredging, 33 ft. For all three alternatives, dikes would be built in two stages, with the first lift being sufficient to accommodate the backlog dredging. The dikes would then be raised to their final height. For Alternative 1, the dikes would be constructed to 30 feet, as specified in the DDR, giving some additional storage capacity over and above the stated project requirements and extending the life of the facility. Disposal of sediments classified as TSCA sediments would take place in a subcell on the north side of the facility for all three alternatives. Before TSCA sediments are dredged, non-TSCA sediments will be placed in the designated subcell and allowed to consolidate, providing at least 3 feet of non-TSCA material underlying the TSCA sediments. Following placement of the TSCA sediments, additional lifts of non-TSCA material will also be placed over the TSCA material, further encapsulating the material. The entire facility will then be capped in accordance with the design specifications contained in the DDR.

Wastewater Treatment

All water produced at the disposal site must be treated in a WWTP constructed on the site. This includes effluent, consolidation, runoff, and groundwater flows generated by the containment system required under conditions of RCRA site closure. The largest design flows are groundwater production during drawdown, storm flows, and hydraulic dredge effluent. The rate of drawdown will be determined by the amount of time available to achieve the required drawdown. Allowing for infiltration from storm events, rates were estimated at between 212 gpm and 1389 gpm for a drawdown period ranging from 1-6 months, a drawdown of 2 to 4 feet, and a specific storage of 0.2 and 0.3. WWTP costs will be minimized if pumping is initiated sufficiently in advance of dredging to achieve drawdown at a pumping rate within the WWTP capacity requirements of the selected dredging alternative. Maximum combined flow during hydraulic dredging was estimated to be approximately 2700 gpm, and this is the estimated peak WWTP capacity requirement for the hydraulic dredging alternatives. Because effluent production is low for mechanical dredging, storm flow governs treatment plant capacity requirements for this alternative. With adequate flow equalization, a WWTP capacity of 200 gpm will provide reasonable drawdown capability and is sufficient to accommodate runoff. Based on reservoir analysis, an available storage volume of 135 acre-ft will provide storage adequate for over 99% of the predicted storm flows. This can be accomplished in a 10-acre equalization cell at the specified dike heights. For the hydraulic dredging alternatives, WWTP capacity required to accommodate the dredge discharge will be sufficient to handle expected storm flows. Precipitation events exceeding this amount during the useful life of the facility could be handled by using the CDF as a temporary equalization basin. A 2-acre equalization basin will provide a minimum of 1-day residence time for the purposes of evening out influent flow fluctuations and concentrations for the hydraulic disposal alternatives.

Volatile and Particulate Emissions

Relative flux rates for volatile emissions were calculated for representative conditions at the CDF for both accelerated hydraulic and mechanical dredging scenarios, assuming unit concentrations of the constituents of concern in the sediments and corresponding predicted concentrations in effluent, runoff and consolidation flows. The volatile emission rates for the standard hydraulic dredging scenario would fall between the accelerated hydraulic and mechanical dredging. Equalization basins were assumed to be ponded at all times. The chemistry of the water in the equalization basin would be expected to be dominated by effluent during hydraulic disposal and by runoff and consolidation flows during the off season and during mechanical dredging. When compared, flux rates are similar for mechanical and hydraulic disposal; although, flux rates for constituents having low partitioning coefficients are somewhat higher for the hydraulic alternatives. Particulate emissions (losses of fugitive dust) were also evaluated for the various alternatives. Particulate emissions will be a function of the area present in an exposed, unvegetated, dried condition, and the length of exposure. During periods where precipitation exceeds evaporation, limited fugitive dust is expected from dried sediment surfaces, as is the case during the off season. For mechanical dredging in the off-season, fugitive dust production is estimated to be about 1.6 times as large as for the hydraulic alternative, based on relative exposed areas and surface conditions. Similarly, during the dredging season, fugitive dust for the mechanical dredging alternative is estimated to be about 1.5 times that for hydraulic dredging. Greater particulate losses would also be expected from mechanical material transfer systems than for hydraulic pipeline operations. In general, somewhat greater volatile emissions are expected for hydraulic dredging, and greater particulate losses are expected for mechanical dredging.

Costs

Planning-level capital and operating cost estimates were prepared to provide an economic basis for comparison of the dredging and disposal alternatives. These cost estimates do not represent complete project construction cost estimates, and costs common to the three alternatives were not included. Major elements of the cost estimates included were:

- Underwater survey
- Debris removal
- Dike construction
- Dredging
- Surface water pumping
- Suspended solids removal (flocculation) in the CDF
- Wastewater treatment
- Dredged material management

Unit cost estimates were either taken from the DDR or developed for the specified cost items. All costs were adjusted to a January 2003 basis and included both direct and indirect costs and contingency. Present value comparison of the alternatives indicates that alternative 1, mechanical dredging, is the least cost alternative, with alternative 2 next highest, and alternative 3 the highest cost alternative.

Major Findings

- Hydraulic dredging is feasible, with completion of backlog dredging in a period of 4 years, and some modification of the existing proposed CDF design developed for mechanical dredging in the DDR.

- Treatment of wastewater to permit discharge to the IHC is feasible. Preliminary plant designs and cost estimates were developed for comparison. Hydraulic dredging requires greater capacity and higher volumes of water must be treated during dredging.
- Relative air and particulate emissions are not expected to be greatly different for mechanical and hydraulic dredging alternatives. Vegetative controls are expected to be feasible to reduce particulate emissions if this is determined to be necessary.
- The least cost alternative based on the present value analysis is Alternative 1.

Future Work/Issues

The following items are recommended for further consideration as the project progresses to the detailed design phase:

- Re-evaluate equalization basin requirements for hydraulic dredging alternatives in consideration of potential cost savings to be realized by decreasing standby WWTP capacity in the non-dredging season
- Conduct additional hydraulic analysis to optimize pump and equilibrium basin size, and evaluate impact on concentrations of SS from the equalization basin
- Conduct settling tests to design and evaluate appropriate coagulant treatment, and evaluate expected SS concentrations following secondary settling
- Further evaluate requirements and constraints associated with vegetating the site for particulate control

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Appendix A - CDF Sizing

Calculation of Available Sediment Storage Area

Available sediment storage area was calculated based on available area within the CDF dikes, at ½ the maximum sediment storage depth, as determined by settling and consolidation analysis. Figure A1 illustrates the relative dimensions utilized in the calculation.

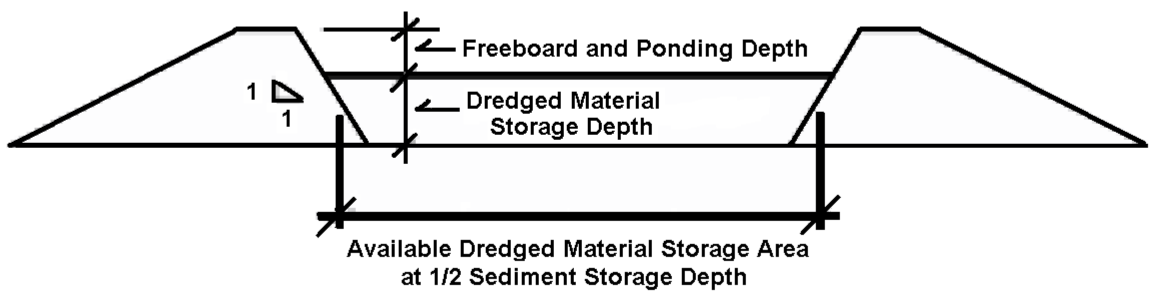

Figure A1. Available storage area at ½ sediment storage depth

Calculation of Placement Depths – Hydraulic Dredging

Initial dredged material depth at the end of each placement period was estimated using SETTLE. Normally, a CDF will be designed to provide sufficient capacity to accommodate the bulked volume of sediments to be dredged, sufficient hydraulic retention time to permit clarification to occur (reduction in slurry solids concentrations from influent to effluent), and sufficient ponding area and depth to reduce effluent SS to acceptable levels during primary settling in the disposal cell. Depending upon the sediment characteristics and operating conditions, any one of these conditions may dictate the maximum CDF capacity. In this case, secondary settling and treatment will be utilized to further reduce effluent SS and contaminant concentrations. CDF design was therefore largely governed by clarification and storage volume requirements, taking into account levels of SS carrying over into the secondary settling basin. The analysis conducted with SETTLE can be repeated, varying such input parameters as available storage area, until an acceptable compromise between lift depth and effluent SS solids is achieved. The following discussion was extracted directly from the SETTLE program documentation (Hayes and Schroeder 1992b) and explains the general design rationale for these three elements.

Design for Initial Storage

A CDF designed for a single dredging project must be capable of storing the dredged material for a particular disposal activity at its largest volume. This occurs just as disposal ends and is commonly referred to as the initial storage volume. The initial storage volume for the dredged material depends upon many aspects of the dredging project, including sediment characteristics (primarily the fraction of fines versus the fraction of sands), settling characteristics of the material, volume of sediment to be dredged, and disposal rate. The required CDF volume, however, is significantly larger than just the initial volume of sediment to be dredged, particularly for hydraulic dredging transport or disposal projects. The CDF must also include additional depth for ponding to allow sedimentation during the final, and most critical, stage of dredging and dike freeboard above the highest anticipated water surface.

SETTLE determines the average concentration of settled solids within the CDF at the end of disposal, referred to as the design concentration, using the compression settling test data. The design concentration is used to calculate the initial storage requirements for the fine-grained (smaller than No. 200 sieve) fraction of the dredged material. Coarse-grained (larger than No. 200 sieve) material behaves independently and much differently. Thus, the storage volume required for the coarse-grained material is determined based upon the input data and then added to the volume of fine-grained material to yield the minimum initial storage required. The volume is not an estimate of the long-term needs for multiple-disposal activities; it is the volume required for the single disposal project under consideration.

Because of siting, construction, and permitting complications, CDFs are frequently sized to receive multiple disposals. The number and frequency of disposals depends upon the location and dredging projects being served. Under these circumstances, initial storage volume will likely become a constraint only as the CDF ends its service life. Estimates for long-term storage capacity can be made using the consolidation and desiccation (PSDDF) module of ADDAMS (Stark 1996).

Initial storage volume can be the controlling design factor regardless of the settling behavior exhibited by the material. In the unusual case that the material exhibits compression settling at a concentration at or below the expected inflow concentration, the design for initial storage might well be the only design consideration required.

Design for Clarification

Sediments containing saline pore water (>3 ppt salt concentration) frequently settle as a tightly formed soil matrix restricted by the upward flow of water through the matrix. The result is a clarified supernatant above a well-defined soil-water interface that continuously settles downward at a slow rate. This is commonly referred to as zone settling. Besides occurring in salt-water sediments, zone settling may occur in freshwater sediments if solids concentrations are high enough or if the particle surface characteristics are flocculent enough. If the dredged material exhibits zone settling behavior at the expected inflow concentration, the zone settling test results are used to calculate the required ponded surface area in the CDF for effective zone settling (clarification). This surface area is the minimum area required to remove suspended solids from the surface layers at a rate sufficient to form and maintain a clarified supernatant that can be discharged.

Flocculent settling occurs in the supernatant water above the interface and controls the quality of the supernatant. Suspended solids concentration in the clarified supernatant varies widely between sediments but is generally on the order of 50 to 500 mg/l. Additional calculations using flocculent settling data for the solids remaining in the ponded supernatant water are required to design the CDF for a specific effluent quality standard for suspended solids. These calculations are identical to those described below.

Design for Effluent Quality

The concentration of effluent suspended solids depends on the flocculent settling characteristics of the sediment, the depth from which fluid is withdrawn at the weir, and hydraulic retention time within the CDF. Because of the low viscosity of the supernatant water and the high flow rates commonly found in CDFs, the withdrawal depth is essentially equivalent to the ponded water depth. SETTLE uses the average ponded depth to estimate the residence time and compute the average solids concentration in the discharge. A minimum average ponded depth of 2 ft is usually recommended for the design; greater depths of ponding reduce the surface area required for adequate solids removal. For most cases, constant ponded depth can be maintained by raising the weir crest and, hence, water surface elevation as settled material accumulates in the CDF. CDFs constructed in a rectangular shape with sides as near the same length as possible minimize the dike length required for a given surface area or storage volume. This typical shape combined with point source inflows result in an actual hydraulic retention time considerably less than the theoretical retention time. Thus, calculating the effluent suspended solids concentration requires correcting the theoretical retention time for hydraulic inefficiencies. Actual mean hydraulic retention time and hydraulic efficiency for a given flow rate, design area, and ponding conditions and the theoretical residence time can be estimated using the DYECON module (Hayes and Schroeder 1992a).

Calculation of Placement Depths – Mechanical Dredging

Initial placement depth of mechanically dredged material assumes that the material will expand in response to the change in effective stress when the material is removed from the in situ environment. A representative bulking factor was assumed, and the bulked sediment lift depth calculated as follows:

$$D = \left(\frac{V}{e_i + 1} \right) * \frac{(1 + e_b) * 27}{(A * 43560)}$$

where

D = lift depth of bulked sediment, ft
V = in-situ volume of sediment, cy
e_i = in-situ void ratio
e_b = bulked void ratio
A = available storage area in cell, acres

Bulked void ratio is calculated using:

$$e_b = (e_i + 1) * F_b - 1$$

where

F_b = assumed bulking factor, in this case 1.1. This factor was derived from empirical data obtained from other projects, and professional judgment.

Consolidation Analysis

CDF sizing must take into account changes in the depth of material placed in the CDF over time. The following was extracted from the documentation for the PSDDF model (Stark 1996), which was used to estimate the consolidation of the IHC dredged material after placement in an upland CDF.

The height of the dredged fill is reduced by sedimentation, primary consolidation, secondary compression, and desiccation. Primary consolidation, secondary compression, and desiccation are accounted for in the microcomputer program **P**rimary consolidation, **S**econdary compression, and **D**esiccation of **D**redged **F**ill (PSDDF). The sedimentation process is complete shortly after material deposition and therefore is not included in PSDDF because it has little, if any, effect on the long-term capacity of a placement area. (Initial sediment height resulting from the sedimentation process is used as an input parameter to PSDDF, and is estimated using either SETTLE, for hydraulically dredged sediments, or the bulking factor, for mechanically dredged sediments, as described in the preceding section). Tests to ascertain the sedimenting nature of a material and procedures for calculating the effects on placement area filling are described in EM 1110-2-5027 (Headquarters, Department of the Army 1987). The three most important natural processes affecting the long-term height of confined dredged material are primary consolidation, secondary compression, and desiccation. Many fine-grained dredged materials may undergo strains greater than 50% during self-weight consolidation. Greater strains are possible, if the placement area is managed so that the surface water is removed and desiccation can occur. The resulting problem is to determine the time rate of settlement for dredged material subjected to the effects of (a) self-weight consolidation, (b) secondary compression, (c) crust formation caused by desiccation, and d) additional consolidation due to the surcharge created by the desiccated crust.

PSDDF simulates the primary consolidation, secondary compression, and desiccation processes in fine-grained soils (e.g. dredged fill) using the one-dimensional finite strain theory of consolidation (Gibson et al. 1967), the secondary compression theory proposed by Mesri and Godlewski (1977), and an empirical desiccation model (Cargill 1985). PSDDF calculates the total settlement of a dredged fill layer based on the consolidation characteristics of the soils above and/or below the layer, the consolidation characteristics of the dredged fill, local climatological data, and surface water management techniques within the containment area. This settlement is then accumulated for each compressible layer within the area, and a cumulative settlement for all dredged fill and compressible foundation layers is calculated. Additional layers of dredged fill can be added at any time.

The major input required by PSDDF is the void-ratio effective stress and void ratio permeability relationships obtained from laboratory consolidation tests on the dredged fill and compressible foundation materials. The recommended laboratory testing procedures to obtain these relationships are described by Cargill (1985 and 1986). In addition, specific gravity of solids, initial void ratio, C_α / C_c, ratio where C_α is the secondary compression index and C_c is the compression index, C_r / C_c ratio, where C_r is the recompression index, and the desiccation characteristics of the dredged fill material are required. The values of C_α, C_c, and C_r, can be obtained from a one-dimensional oedometer test performed in accordance with ASTM (1993) Standard D2435-80. Climatological data, anticipated dredging schedules and quantities, water table elevation, and drainage characteristics of the containment site are also required. Equations used for the previously described calculations can be found in Stark (1996).

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Appendix B - WWTP Flows and Concentrations

Estimating Contaminant Concentrations in Aqueous Streams

Constituent concentration in each of the flows making up the combined flow to the WWTP was estimated based on historical data, effluent testing, and partitioning theory. All available data was evaluated. Data considered to be most reliable and representative was selected and used in estimating dissolved concentration of constituents in combined flows to the WWTP, using the equations in the sections following. Data obtained directly from groundwater, and elutriate test and runoff analysis, was input directly into the equations. For constituents where such data was unavailable or appeared to be unreliable, dissolved concentrations were estimated based on partitioning theory. Estimation of effluent, porewater and runoff concentrations based on partitioning theory requires the following information: sediment concentration (mg/kg or ug/kg), sediment or slurry character (grain size, specific gravity, percent organic carbon, percent solids), site water character (dissolved organic carbon), and partitioning coefficients for the constituents of concern. Partitioning coefficients reflect the distribution of contaminant between the solid phase and the aqueous phase, for a soil or sediment in contact with water, or containing pore water. Partitioning coefficients are obtained from the literature and recognized databases, or calculated using accepted theoretical relationships.

For inorganic contaminants and organotins, the K_D 's are selected from literature sources, past elutriate testing, field observations and other empirical evidence (Schroeder et al. in Preparation). For organic contaminants (PAHs, organophosphorus pesticides, chlorinated pesticides, semivolatile organic compounds, PCBs and dioxins), K_D is calculated as follows:

$$K_D = \frac{0.617 * FOC * K_{OW}}{1 + \left(0.617 \times 10^{-6} DOC * K_{OW}\right)}$$

where

FOC = fraction organic carbon in the solids
K_{OW} = octanol-water equilibrium partitioning coefficient, mg/kg
DOC = dissolved organic carbon concentration, mg/l

Where sediments are hydraulically dredged, the effluent produced is a result of mixing material from two systems initially at equilibrium (sediment/porewater and water column/suspended solids), in which the contaminants present are partitioned between the solids and the surrounding fluid (Schroeder et al. in Preparation). When these systems are mixed, a new state is created that is bounded by theoretical conditions: 1) the contaminants in the mixture will establish a new equilibrium between the solids and the water, or 2) dissolved concentrations in the effluent will be a simple function of mixing the pore water and the carrier water.

The retention time in most CDFs is on the order of one day to a few days. Contaminant partitioning between the solid and aqueous phases in the influent slurry is not likely to reach equilibrium due to the short contact time after mixing of in-situ sediments with carrier water, and the limited oxidation occurring in the influent slurry. Equilibrium partitioning is therefore considered to be a boundary condition for effluent quality, and a screen based on equilibrium partitioning would therefore be conservative. Effluent quality predicted by simple mixing provides the second boundary condition.

Consolidation flows are a mixture of pore water and carrier water that has been mixed with the sediment during dredging. Here also, equilibrium is a conservative assumption of contaminant concentration, but is more likely than for effluent, given the greater length of time that the water is in contact with the solids. Measured pore water concentrations are typically consistent with pore water concentrations predicted using the assumption of equilibrium, and are used as a check on the estimated calculations.

Concentrations in groundwater flows result from contact of water infiltrating through the soil to a groundwater layer, and contact of the groundwater with soil solids and non-aqueous phase contaminants. Pore water concentrations are typically considered to represent a conservative estimate of groundwater concentrations, although in some cases, higher concentrations are reported as a result of the influence of non-aqueous phase contaminant sampling. Where non-aqueous phase contaminant (pure product) was obtained during groundwater sampling, reported or predicted groundwater concentrations may exceed contaminant solubility.

Concentrations in runoff result from re-suspension of solids from the dredged material surface, and partitioning between the water and solids while they are in contact. While particulate concentrations can be high in runoff, dissolved concentrations may be lower than for effluent or consolidation flows, assuming a more limited contact time with the solids. If the material has become oxidized, however, higher dissolved concentrations of some metals may be seen in runoff than in effluent.

Estimating Volumetric Flows to WWTP

Precipitation

Net infiltration (or runoff) was estimated by generating 40 years of precipitation and evaporation data for the area using the HELP model (Schroeder et al. 1994). Results were divided into dredging and non-dredging seasons, and average values calculated from the seasonal data. Net infiltration values are the same for all alternatives, for the same periods of consideration. Except during the drawdown period, when the CDF is empty and all precipitation was assumed to infiltrate to the groundwater, excess infiltration was assumed to occur and it would be handled as runoff.

Consolidation

Consolidation flows were estimated based on the change in dredged material volume reflected by the consolidation analysis conducted using PSDDF, which is further described in Appendix A. This approach assumes a saturated material. Any change in volume is assumed to be equivalent to the volume of water released from the material.

Effluent

Effluent flows to the WWTP were calculated based on an assumed production rate for a 12-inch hydraulic dredge, and a representative slurry solids content. While the dredge was assumed to operate for 14 hours per day, 6 days per week, effluent was assumed to be pumped from the equalization basin at a lower rate, 24 hours per day, 7 days per week, thus equalizing flows to the WWTP. Little or no effluent flow is expected from mechanically dredged materials.

Groundwater

Groundwater flows are given in the DDR for gradient maintenance. Groundwater flows were estimated for the drawdown period based on the volume of water to be pumped and the interval over which it was to be pumped, taking into account additional pumping requirements due to expected infiltration from precipitation. Groundwater flows will be highest during drawdown, when a groundwater gradient is being established. Highest infiltration will also occur during this period, while the CDF is empty. After dredged material is placed in the CDF, infiltration is expected to be minimal. For purposes of this evaluation, 100% of excess precipitation (precipitation less evapotranspiration) was assumed to infiltrate during the drawdown period. 100% of excess precipitation was assumed to run off (0% net infiltration) after material was placed in the CDF. Volumetric flow to the WWTP is the same for both assumptions, as both groundwater and runoff will be pumped to the WWTP.

Estimating Characteristics of Combined Flows

WWTP flows are made up of combined aqueous streams resulting from dredge discharge, consolidation, precipitation runoff, and groundwater pumping. The relative proportion of each of these flows varies seasonally and from year to year, and with project phase and dredging activity. The volume and character of the combined flows was estimated for dredging and non-dredging seasons. The volumetric flow going to the

WWTP is simply the sum of the average flows for each stream for the period being considered:

$$Q_{combined} = Q_{groundwater} + Q_{runoff} + Q_{effluent} + Q_{consolidation}$$

where

Q_i = volumetric flow of waste stream i, gpm

The concentration of the combined flows is a function of the relative volume and contaminant concentrations in each flow. A simple mixing equation is used to estimate contaminant concentration in combined flows:

$$C_{i_{Combined}} = \frac{C_{i_{GW}} Q_{Groundwater} + C_{i_R} Q_{Runoff} + C_{i_{Eff}} Q_{Effluent} + C_{i_{Cons}} Q_{consolidation}}{Q_{combined}}$$

where

C_{ij} = concentration of constituent i in the flow j, mg/L

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Appendix C - Volatile Emissions

When contaminated dredged material is placed in a CDF, volatilization of chemicals associated with the sediment is an environmental concern. The following discussion was extracted from Myers and Schroeder (In Preparation), describing screening level procedures for estimating volatile emissions from dredged material. There are presently no laboratory tests available for predicting volatile emissions from ponded water in CDFs. Methods for predicting volatile losses from exposed dredged material and water ponded over dredged material have been developed (Thibodeaux 1989; Valsaraj et al. 1997, 1999; Price 2000). Estimates of volatile contaminant fluxes are based on models using site-specific data, such as water or sediment contaminant concentrations, and accepted values for constituent specific parameters, such as Henry constants and chemical diffusivity in air.

Volatile emissions are estimated for two conditions in the CDF: exposed sediment and ponded water in contact with the sediment. The rate of volatilization from exposed sediment is affected by geotechnical properties of the dredged material, such as porosity and water content, chemical diffusivity in water and air, air-water-solids partitioning, and wind speed and temperature (Myers and Schroeder In Preparation). For ponded conditions, contaminants must dissolve into the water from the solids, and then be transported to the surface in order to cross the air-water interface. Suspended solids control dissolved contaminant concentrations during active filling. When suspended solids concentrations diminish, bottom sediments will continue to contribute to the water column concentrations, although typically concentrations will be lower than during filling operations. Pore water released as the dredged material consolidates may also make a significant contribution for a time. The rate of volatilization from ponded water is controlled by dissolved chemical concentration, Henry's constant, and wind speed (Myers and Schroeder In Preparation).

Screening level volatilization calculations assume equilibrium between dissolved contaminants and contaminants sorbed to solids. For exposed dredged material, chemical emissions are assumed to begin as soon as the dredged material is exposed to air, and flux rate is controlled by the mass transfer rate between air and water, with air-side resistance governing (Myers and Schroeder In Preparation). After concentrations in the surface layers are depleted, further releases are limited by vapor diffusion on the dredged material side, and volatilization rates decrease. The screening level calculations estimate the initial volatilization rate, which is considered to be conservative. The applicable flux equations are given below.

Ponded Conditions

$$N = \frac{C_W \, K_{OL}}{10^9}$$

where

N = flux through the air-water interface, g/cm^2/s
C_w = controlling dissolved chemical concentration, ug/L
K_{OL} = overall liquid phase mass transfer coefficient, cm/s

The liquid phase mass transfer coefficient is calculated as follows (Liss and Slater 1974; Thomas 1990; Thibodeaux 1996):

$$\frac{1}{K_{OL}} = \frac{1}{K_L} + \frac{1}{H \, K_G}$$

where

H = Henry's constant, dimensionless
K_G = gas-side mass transfer coefficient, cm/s
K_L = liquid-side mass transfer coefficient, cm/s

The gas-side mass transfer coefficient is given by (Thomas 1990):

$$K_G = 0.32 \left(v_x + v_{curr} \right) \sqrt{\frac{18}{MW}}$$

where

v_x = wind speed, m/s
v_{curr} = water velocity in the CDF, m/s
MW = molecular weight of the chemical, g/mol

The liquid side mass transfer coefficient, for wind speeds greater than 1.9 m/s and less than 5 m/s is given by (Thomas 1990):

$$K_L = 0.0065 \left(\frac{v_{curr}^{0.969}}{Z^{0.673}} \right) \sqrt{\frac{32}{MW}} \; e^{0.526 \, v_x^{-1.9}}$$

where

Z = ponded water depth, m

Exposed Sediment Conditions

$$N = 2 \left[\frac{\dfrac{q_{sl} H}{1000\, K_d}}{\dfrac{1}{K_G} + \sqrt{\dfrac{\pi t_c}{D_a \left(n_a + \dfrac{K_d}{S} \right)}}} \right]$$

where

q_{sl} = sediment contaminant concentration, mg/kg
K_d = partitioning coefficient, L/kg
N = average volatile flux over the critical time t_c, g/cm^2/sec
D_a = diffusivity in air, cm^2/sec
n_a = air filled porosity of the drying dredged material, default = 0.2, dimensionless
S = solids concentration of settled solids, kg/L
t_c = critical exposure period during drying, sec

The critical exposure period is user defined and determines the period over which the average flux is calculated. For the purposes of this study, 16 days was assumed, which is the average duration between significant wetting events. The model assumes background air concentrations are zero, which is conservative, as the presence of air concentrations greater than zero reduces volatile flux.

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Appendix D - Comparative Cost Analysis

The comparative cost analysis does not represent complete project construction costs. Construction features common to all three dredging alternatives are not included in the comparative dredging cost analysis. The common features are:

- Railroad relocation
- Cutoff wall
- Hydraulic gradient control system
- Cap on southern end of site and around perimeter of site
- 3' clay liner underneath the dike walls
- Air monitoring
- Certain O&M costs

The comparative cost analysis is made up of over twenty individual estimates. The individual estimates are divided between three categories: 1) construction and dredging activities, 2) operation and maintenance (O&M) activities, and 3) markups and contingencies. The construction and dredging estimates were either taken from the DDR or were developed specifically for the comparative analysis. In some cases, the unit prices from the DDR were applied to increased quantities. Any costs based on the DDR estimates were adjusted to January 2003 dollars. Table D1 shows the source for each individual estimate. The components of the comparative analysis are described in more detail in this appendix.

Construction and Dredging Activities

Underwater Survey/Debris Removal Costs

C&C Technologies provided a quote for the underwater survey. The activities included consisted of mobilization/demobilization to the site, performing the fieldwork and writing a final report. The personnel and equipment identified for the site work included survey vessel, global position system, side-scan sonar, sub-bottom profiler, cesium magnetometer, field project manager and senior field personnel.

It was assumed a mechanical dredging plant would do the debris removal operation. The amount of debris is not known at this time, but it was assumed it would take 70 days. The debris removal is only associated with the hydraulic alternatives. For mechanical dredging, debris removal can be efficiently completed during the actual dredging. The mechanical dredge rate has been adjusted to account for debris removal as needed.

Table D1. Sources of Individual Cost Estimates			
	Alternative 1 Mechanical	**Alternative 2 Hydraulic**	**Alternative 3 Accel Hydraulic**
Construction and Dredging Activities			
Underwater survey	N/A	Vendor quote	Vendor quote
Debris removal	N/A	New estimate by Corps	New estimate by Corps
Year 1, cell construction,	DDR (unit price)	DDR (unit price)	DDR (unit price)
Year 1, additional clay liner,	N/A	DDR (unit price)	DDR (unit price)
Year 2, cell construction,	DDR (unit price)	DDR (unit price)	DDR (unit price)
Year 2, additional clay liner,	N/A	DDR (unit price)	DDR (unit price)
Standpipes/weirs	New estimate by Corps	New estimate by Corps	New estimate by Corps
Pumps in standpipes	New estimate by Corps	New estimate by Corps	New estimate by Corps
Raise dike heights	DDR (unit price)	DDR (unit price)	DDR (unit price)
Wastewater treatment plant (WWTP)	New estimate by MWH	New estimate by MWH	New estimate by MWH
Dredging	DDR (unit price)	New estimate by Corps	New estimate by Corps
Pumps during dredging	N/A	New estimate by Corps	New estimate by Corps
Cap	DDR (lump sum)	DDR (lump sum)	DDR (lump sum)
Operations and Maintenance Activities			
O&M of WWTP	New estimate by MWH	New estimate by MWH	New estimate by MWH
O&M for pumps in standpipes	New estimate by Corps	New estimate by Corps	New estimate by Corps
CDF surface management O&M	N/A	N/A	New estimate by Corps
DDR (unit price or lump sum) based on unit cost or estimate presented in USACE, Chicago (2000); adjusted to current dollars.			

Dike Construction Costs

The dike construction costs are split into several individual estimates - clay liner (beneath and on the inside of the dikes); dike wall (cell) construction; raise dike heights; and cap. A brief description of the items follows. The initial dike wall construction will be phased over 2 years with several cells being built the first year and the remaining cell(s) the second year. The cost for each year is itemized in the cost estimate and referred to as “Cell Construction” in Table D1. The item “Raise Dike Heights” in Table D1 refers to the stage two construction of the CDF when the dike walls will be increased from the initial height to the final height. Stage two construction will occur after backlog dredging is complete and before maintenance dredging begins. The item labeled "Cap" in Table D1 refers to the cap that will be placed over the CDF approximately two years after completion of dredging.

The dike design in the DDR specifies a three-foot clay liner underneath the base of the dikes and on the interior face of the dikes. Higher dike walls will be required for hydraulic dredging than for the mechanical dredging alternative, resulting in a larger cross section and wider base. The hydraulic dredging alternatives will require a larger quantity of clay to construct the liner. This additional cost is referred to as the "Additional Clay Liner" in Table D1 and is also itemized by year of construction in the cost estimate.

Dredging Costs

The mechanical dredging operation was originally estimated in the DDR. The operation activities consist of removing the sediment by clamshell bucket, placing the sediment in scows, and towing the scows to the site. Using a land-based crane, the sediment will be unloaded from the scows and into trucks. The trucks will unload the sediment into CDF, using the dike as a haul road. Annual mob/demob costs were also incorporated in the cost estimate.

The hydraulic dredging operation includes the cost for a 12-inch hydraulic dredge, supporting launch, barges, pipeline and booster pumps, operators and deck hands. It was assumed on average that two booster pumps would be required along the length of the pipeline over the project lifetime. Annual mob/demob costs were also incorporated in the cost estimate.

Surface Water Pumping Costs

Pumping is required to maintain the groundwater gradient and to transfer surface water accumulated in the disposal cells from dredging, consolidation, and precipitation to the wastewater treatment plant by way of the CDF equalization basin. Groundwater pumping requirements are assumed to be the same for all alternatives, and are not included in this comparative analysis. Pumping rates from the disposal cells will vary, depending upon the time of year and dredging activity. Pumping requirements will be highest during hydraulic disposal, and lowest during non-dredging periods.

There will be a separate set of pumps to handle the two flow situations. The two sets of pumps are:

1. Standpipe/weir pumps
2. Transfer pumps

Table D2 summarizes the average pump flow rates and duration for both of the pumping systems. The standpipe/weir pumps, referred to as "Pumps in Standpipes" in Table D1, consist of 5 standpipe/weirs and 5 pumps which will operate year round on an as-needed basis to handle storm flows. Four pumps, one per cell, will transfer water from the primary cells to the CDF equalization basin and one pump will transfer water from

Table D2. Pump Flow Rates and Pumping Duration

Alternative	Avg. Duration	Transfer Pumps for Cells	Equalization Basin Pump
Fixed Standpipe/Weirs & Pumps (Non-Dredging)			
Alternative 1 (Mechanical)	32 years	2 @ 300 gpm (Cells 2 & 3); 1 @ 200 gpm (Cells 1); 1 @ 100 gpm (TSCA Cell) Operating 33 days for all pumps Standby 332 days/yr	1 @ 200 gpm Operating 146 days/yr Standby 219 days/yr
Alternative 2 (Hydraulic)	32 years	3 @ 300 gpm (Cells 1, 2 & 3); 1 @ 100 gpm (TSCA Cell) Operating 33 days for all pumps Standby 332 days/yr	1 @ 200 gpm Operating 163 days/yr Standby 202 days/yr
Alternative 3 (Accelerated Hydraulic)	3 years	3 @ 300 gpm (Cells 1, 2 & 3); 1 @ 100 gpm (TSCA Cell) Operating 35 days for all pumps Standby 330 days/yr	1 @ 200 gpm Operating 174 days/yr Standby 191 days/yr
	23 years	3 @ 300 gpm (Cells 1, 2 & 3); 1 @ 100 gpm (TSCA Cell) Operating 33 days for all pumps Standby 332 days/yr	1 @ 200 gpm Operating 163 days/yr Standby 202 days/yr
Floating Weirs & Transfer Pumps (During Dredging)			
Alternative 1 (Mechanical)	N/A	N/A	N/A
Alternative 2 (Hydraulic)	25 years @ 51 days/yr	2 @ 1550 gpm Operating 35 days/yr Standby 16 days/yr	1 @ 2700 gpm Operating 40 days/yr Standby 11 days/yr
Alternative 3 (Accelerated Hydraulic)	3 years @ 189 days/yr	2 @ 1550 gpm Operating 134 days/yr Standby 55 days/yr	1 @ 2700 gpm Operating 154 days/yr Standby 35 days/yr
	16 years @ 45 days/yr	2 @ 1550 gpm Operating 29 days/yr Standby 16 days/yr	1 @ 2700 gpm Operating 33 days/yr Standby 12 days/yr

the equalization basin to the wastewater treatment plant. The pumps will operate every year, in dredging and non-dredging years. The estimate includes the capital costs for purchasing and installing the pumps. A separate estimate was done for the operation and maintenance of these pumps and is discussed later in the text. Table D2 was used to perform the cost estimates for surface water pumping.

The floating weir and transfer pumps, referred to as “Pumps During Dredging” in Table D1, are for the hydraulic alternatives only. The system consists of 3 pumps located on barges. Two pumps with a combined capacity of 3100 gpm in the disposal cell transfer water to the CDF equalization basin. The pump located in the CDF equalization basin has a capacity of 2700 gpm and transfers water to the wastewater treatment plant. The pumps will be in place and operate during the dredging season only; no cost will be incurred during non-dredging periods. The number of days in operation and number of days in standby status are based on the average length of the dredging projects assuming an average condition for dredge production (a 12-inch cutterhead dredge operating 6 days per week, 14 hours per day, with two booster pumps).

As discussed in the report, flocculant addition is necessary for the hydraulic dredging alternatives to enhance secondary settling in the CDF equalization basin, thereby reducing the suspended solids in the CDF effluent. It is anticipated that flocculants would be introduced at the transfer pumps; the cost of flocculants was therefore included in the transfer pump estimates. Chemical cost of flocculant was assumed to be 10 cents per cubic yard of in situ sediment. Costs for injection system were not included but should be similar for all alternatives.

Wastewater Treatment Plant Costs

The Chicago District contracted Montgomery Watson Harza (MWH), a leader in wastewater treatment plant design and construction, to develop the cost estimate for the wastewater treatment plant. The wastewater treatment plant consists of the following unit processes: inlet surge tank, coagulation and precipitation, clarification, biological aeration, upflow biofilter, zeolite filer and granulated activated carbon filter. A wastewater treatment plant peak flow rate of 2,700 gallons per minute (gpm) was assumed for the two hydraulic dredging alternatives and a peak flow of 200 gpm for the mechanical dredging alternative.

Major unit processes and preliminary sizing calculations were performed using design criteria based on accepted, published industry standards, professional experience, selected design flows and concentrations (Tables 6, 7, and 8 in the main report) and additional design criteria contained in Table D3. The WWTP cost estimate includes necessary pumps, accessory tanks, and chemical feed systems for a complete system. The estimate is summarized in MWH’s report titled “Capital and OM&M Cost Estimate Wastewater Treatment Plant”.

Table D3. General Design Criteria for Flow from CDF to WWTP				
	BOD_5/TOC Ratio	**BOD_5/NH_3 Ratio**	**Nitrification F/M**	**CDF Discharge TSS (mg/L)**
Design Conditions for all 3 alternatives	1.80	2.7	0.5	80

Cost Estimate Basis

The WWTP cost estimate is based on a conceptual design. At the conclusion of a conceptual estimate done by MWH, the costs are evaluated and archived for use as a basis for future estimates that have like parameters. MWH drew from the most recent archives (within last 2 years) of estimates that shared common parameters.

In addition to previous estimating data, MWH maintains a pricing database that is closely aligned with R.S. Means, Richardson's, and other industry pricing sources. MWH drew upon these resources to further develop the estimate.

Accuracy Basis

MWH adopted the Association for the Advancement of Cost Engineering's (AACEI) accuracy classification system. The AACEI defines the wastewater treatment plant as a class 4 estimate, which is defined as:

> "Traditionally, Engineering is from 1 to 5% complete, and would comprise at a minimum the following: plant capacity, block schematics, indicated layout, process flow diagrams for main process systems, etc. Typical accuracy ranges for Class 4 estimates are from +/- 15 to 50% (sometimes higher), depending on the technological complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination. Class 4 estimates virtually always use stochastic estimating methods such as equipment factors, Lang factors, Hand factors, Chilton factors, Peters-Timmerhaus factors, Guthrie factors, the Miller method, gross unit costs/ratios, and other parametric and modeling techniques. (Source: Cost Engineering Vol. 39/No. 4, April 1997)"

Subcontractors

For this estimate, MWH combined subcontractor's labor and material into the prime contractor's cost. Most prime contractors will not have the resources to self-perform all the construction work and will subcontract out some of the work. Typical subcontractors will include the following:

- Electrical & Instrumentation Subcontractor
- Mechanical Piping Subcontractor
- Building Erector Subcontractor
- Asphalt Subcontractor

Depending on market conditions, some prime contractors may seek out additional subcontractors such as earthwork or concrete subcontractors to name a few. The subcontractor plan would depend on the staffing load, local labor market, and capabilities of the prime contractor.

Trade Labor Rates

MWH has prepared this estimate using Lake County, Indiana prevailing wage rates for the trade labor associated with the project. The rates include the appropriate fringe benefits, taxes, and other applicable burdens for that labor.

Overall Pricing Sheet

All costs presented in the detailed portion of the estimate are "raw" costs. Those raw costs are then inserted into a pricing sheet that applies the necessary mark-ups. A further description of each of the mark-ups is included in the Pricing Summary Sheet included with the estimate.

Operation and Maintenance (O&M) Activities

The DDR has a list of general O&M activities that would likely be needed over the pre-closure lifetime of the project. Table D4 is the list as presented in the DDR, pages H-5 and H-6.

Table D4. O&M Activities in DDR

Activities	Subactivities
Wastewater treatment	None
Gradient control system	1. Pumping 2. Analytical testing 3. Sample collection
Groundwater monitoring	1. Analytical testing 2. Sample collection
CDF surface water	1. **Pumping** 2. Analytical testing 3. Sample collection
Erosion control	1. Mowing 2. Repairing
Dredged material management	None
Air monitoring plan	None

The O&M activities are assumed to be the same for all alternatives except for three of the above items: wastewater treatment plant, dredged material management, and CDF surface water pumping. New estimates were prepared for the O&M costs of these items.

O&M Costs for Wastewater Treatment Plant

Montgomery Watson Harza (MWH) developed the O&M costs based on the conceptual treatment plant design components discussed in a previous section. The estimate is summarized in MWH's report titled "Capital and OM&M Cost Estimate Wastewater Treatment Plant". The annual O&M activities for the wastewater treatment plant include:

1. Operating the plant
2. Maintaining the plant and
3. Monitoring the unit processes.

There is an O&M cost estimate for each alternative due to the variation in treatment plant flow rates and annual dredging volumes, in the case of the accelerated alternative.

Annual O&M costs were developed by first assuming typical dosages and calculating the projected chemical quantities and costs for process chemicals. The electrical costs for the major pumps and process equipment items were added. The third major cost element, operating labor, was estimated both on a process-by-process basis and by assuming a shift-based staffing program for the entire wastewater treatment plant as a whole. Costs were added for maintenance parts and labor based on a parametric factoring of the equipment costs for each process. Monitoring costs were added by estimating the frequency and cost of sampling and analytical work for both process control and permit compliance verification. Finally, costs were added for process unit cleaning and sludge disposal, and, when needed, rental equipment.

Seasonal variations in wastewater treatment plant flow rate and character may have a significant impact on costs from the perspective of staffing and treatment requirements. During low flow periods, for example, the number of operators could potentially be reduced. For design and cost estimating purposes, it is necessary to evaluate the variation in volume and character of the influent wastewater stream. Table D5 separates the total estimated annual treatment volumes for the different alternatives into three categories: summer dredging period, summer non-dredging period, and winter non-dredging period. The three categories are distinctly different in terms of TOC and ammonia concentrations, which are the most important constituents from a treatment cost perspective in this analysis. Concentrations for TOC and Ammonia reported in Table D5 were estimated based on the combined flows summarized in Table 6. It was assumed for all alternatives that the total suspended solids (TSS) concentration from the CDF effluent was 10 to 30 mg/L. For the hydraulic alternatives, the TSS concentration is achieved by adding flocculants. The pumping rates shown in Table D5 are annual averages of the entire period of operation for the treatment plant: 32 years for Alternatives 1 and 2 and

26 years for Alternative 3. The rates were used to generate average annual cost estimates for the wastewater treatment.

Table D5. Cost Estimating Criteria for O&M Costs for the WWTP

Alternative/ Period	Average In Situ Sediment Volume (cy)	Flow Volume (gallons)	Duration (days)	Flow Rate (gpm)	TOC (mg/L)	Ammonia (mg/L)
Alternative 1 (Mechanical) – 25 years plus 5 off-years; design flow of 200 gpm						
Summer Dredging Period	190,000	1,992,550	55	25	287	421
Summer Non-Dredging Period		4,637,208	128	25	287	421
Winter Non-Dredging Period		35,319,568	182	135	57	85
Alternative 2 (Hydraulic) – 25 years plus 5 off-years; design flow of 2700 gpm						
Summer Dredging Period	190,000	125,704,400	35	2494	59	114
Summer Non-Dredging Period		15,923,676	148	75	361	528
Winter Non-Dredging Period		41,936,797	182	160	145	212
Alternative 3 (Accel Hydraulic) – 3 years; design flow of 2700 gpm						
Summer Dredging Period	715,000	581,827,782	161	2510	52	104
Summer Non-Dredging Period		16,210,992	22	512	341	499
Winter Non-Dredging Period		44,848,445	182	171	106	156
Maintenance & TSCA Hydraulic (Alt 3) – 16 years plus 5 off-years; design flow of 2700 gpm						
Summer Dredging Period	170,000	112,472,357	31	2520	59	114
Summer Non-Dredging Period		15,923,676	152	73	361	528
Winter Non-Dredging Period		41,936,797	182	160	145	212

O&M for Dredged Material Management

CDF surface management refers to trenching the dredged material in the cells to facilitate dewatering. For the majority of the dredging seasons, a 3 to 3½ foot lift will be placed in each cell. The exception to that will be the first 3 years of backlog dredging during accelerated hydraulic dredging, Alternative 3. The lift during those years will range from 7 to 10 feet. It has been assumed that surface management of the dredged material to enhance dewatering will be common to the three alternatives except for the first 3 years of backlog dredging. An estimate has been included over the timeframe of these 3 years, assuming that an amphibious excavator will be used to put in perimeter and interior trenches in the CDF, as needed.

O&M for Pumps in Standpipes

The pumps in the standpipes will operate year round. The pumps are located in each disposal cell and the CDF equalization basin and will primarily pump surface water resulting from precipitation to the wastewater treatment plant. The annual cost of operating, inspecting, repairing, and replacing the pumps are O&M costs and are included in the estimate referred to as “O&M for pumps in standpipes” in Table D1. The transfer pumps for the hydraulic alternatives described in the pumping section above will be part of the contract of the dredging company. These pumps will be removed annually after the dredging season is over. Since O&M costs will be the responsibility of the dredging contractor, there are no separate O&M costs for these pumps.

Markups and Contingency

Markups

The indirect costs or markups are included in the present value analysis. In the case of the unit prices or lump sum estimates published in the DDR, the indirect costs were incorporated in the unit price or lump sum estimates. For the estimates developed in early 2003, indirect costs have been added to the direct costs. The following indirect costs were assumed: 12% for overhead, 10% for profit, and 1% for bond. The markup percentages are based on historical data from previous projects and audits.

Contingency

Contingency has been included in the present value analysis. A contingency, ranging from 15 to 50% with the majority within the 15 to 25% range, was applied to every item. Table D6 is the guidance provided by ER 1110-2-1302, Civil Works Cost Engineering for selecting contingency:

Table D6. Guidance for Contingency Percentages

Phase of Project Development	Total Project Construction Cost	
	Greater than $10,000,000	Less than $10,000,000
Reconnaissance/Feasibility	20 %	25 %
Design Memorandum	15 %	20 %

Table D7 summarizes the contingency assigned to each individual estimate. The level of design, cost of the individual component, and level of inherent risk associated with each item determined the contingency percent. For the majority of the cases, the guidance was adhered to. Where the cost was close to $10,000,000 for one of the alternatives, it was decided to select the same contingency for all three alternatives for consistency. A contingency of 50% was assigned to the debris removal and O&M for dredged material management. The higher contingency is due to the increased risk because these activities are not as defined as the other activities.

Table D7. Contingency Percentages

Parameter	Alternative 1 Mechanical	Alternative 2 Hydraulic	Alternative 3 Accel Hydraulic
Construction and Dredging Activities			
Underwater survey	N/A	20%	20%
Debris removal	N/A	50%	50%
Year 1, cell construction	15%	15%	15%
Year 1, additional clay liner	N/A	15%	15%
Year 2, cell construction	15%	15%	15%
Year 2, additional clay liner	N/A	15%	15%
Standpipes/weirs	25%	25%	25%
Pumps in standpipes	25%	25%	25%
Raise dike heights	20%	20%	20%
Wastewater treatment plant (WWTP)	15%	15%	15%
Dredging	20%	20%	20%
Pumps during dredging	N/A	25%	25%
Cap	15%	15%	15%
Operations and Maintenance Activities			
O&M of WWTP	25%	25%	25%
O&M for pumps in standpipes	25%	25%	25%
O&M for dredged material management	N/A	N/A	50%